AUG 8 1939

ENGINEERING CONSTRUCTION WEEKLY

ENGINEERING NEWS-RECORD

AUGUST 17, 1939

GROWTH IN SUSPENSION BRIDGE KNOWLEDGE

•

by Leon S. Moisseiff

Sketch of Tacoma Narrows Bridge, which presented an extreme problem in suspension bridge design

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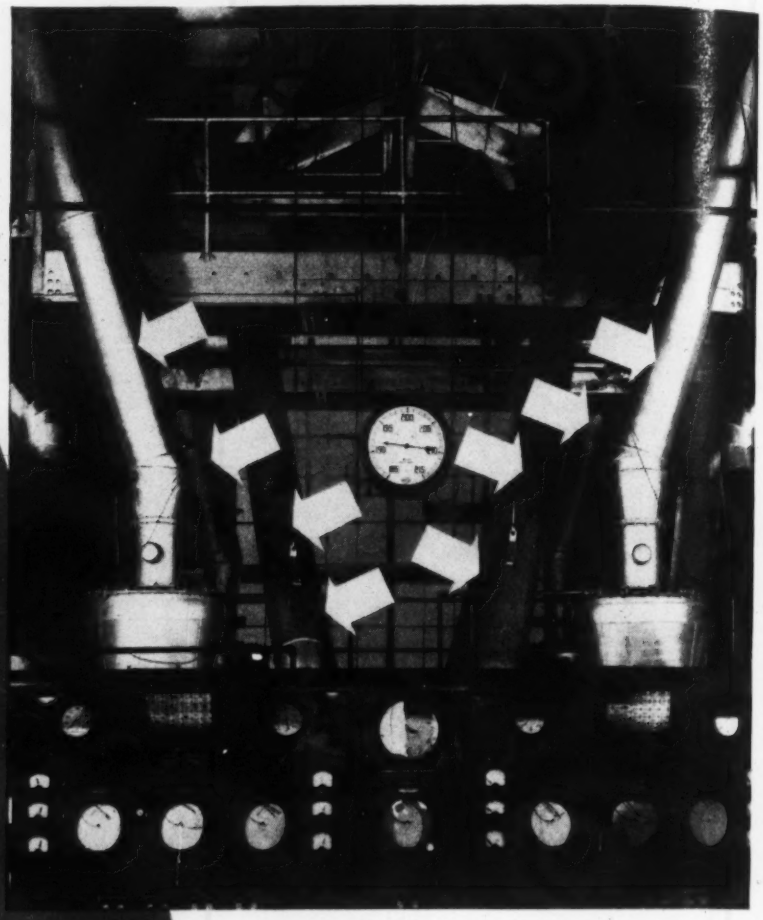
CORROSION COSTS YOU MORE THAN WROUGHT IRON



Especially where Low-Grade Fuels Accelerate Corrosion

Use BYERS WROUGHT IRON

Increasing use of low-grade fuels, made possible by improved combustion equipment and control, has brought a serious problem to the maintenance engineer. Sulphur compounds render the coal, its ashes, and its combustion products all highly corrosive.

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Service records on wrought iron smokestacks that have lasted thirty to forty years first suggested the use of wrought iron for fuel-handling equipment. Today dozens of industrial plants use it for this purpose. Many railroads use wrought iron in all parts of their coaling stations that are subject to marked corrosion. Coal larry hoppers, economizers, pre-heaters, breechings, and similar equipment are made of wrought iron to reduce repair and replacement costs.

The boiler room is only one of hundreds of places where corrosion costs you more than wrought iron. If you want to check on the savings the use of wrought iron can bring, just write us the details of your problem. Our Engineering Service Department will (1) Determine the probable corrosive conditions; (2) Relate these to similar conditions encountered elsewhere; (3) Interpret the results in the light of 75 years' experience with corrosion problems; and (4) Make recommendations . . . supported by service records. Ask, too, for a complimentary copy of our bulletin, "Wrought Iron for Flue Gas Conductors and Coal Handling Equipment."

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ENGINEERING NEWS-RECORD

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THIS WEEK

● Four spans that have had a pronounced effect on the trend in present-day suspension bridge design are discussed in an unusually informative article by Leon S. Moisseiff, dean of the bridge building profession. Mr. Moisseiff tells how each of these spans has contributed to progress in the development of better designs for suspension bridges as requirements became more exacting.

● Henry W. Taylor, New York consulting engineer who has made considerable study of the incineration question, and in this issue presents his opinions on the proper means of making surveys preparatory to the construction of a municipal incinerator. He maintains that there is no universal refuse disposal formula—that each job requires a study of collection methods, along with careful determinations of the quality and quantity of material to be burned.

● Following closely on the heels of his comment-provoking letter on the Atlantic City A.W.W.A. convention, another item by Farley Gannett appears on page 66. In his discussion of "Why So Few Famous Engineers Today?" Mr. Gannett demonstrates that he can turn his analytical mind to subjects more serious than those contained in his convention comment.

THINGS TO COME

THE PROS AND CONS of the effect of the WPA on the construction industry are to be discussed in the Aug. 31 issue by two outstanding men, W. A. Klinger and Col. F. C. Harrington. Readers are to be given an opportunity to express their views on this controversial matter.

BOSTON'S new pressure aqueduct of reinforced concrete pipe will be the subject of two articles in the Aug. 31 issue, one dealing with the reasons for the aqueduct and one telling how the pipe is made and placed.

ANOKA GETS A NEW KIND OF TRAFFIC COP!

● See how this concrete island, made with Atlas White, dominates the whole road at Anoka, Minn. General contractor, McCree & Co., St. Paul. Sub-Contractor, Crown Sidewalk & Blk., Co., Minneapolis. Engineers, Minn. Dept. of Highways.

WHITE CONCRETE ISLANDS—on duty day and night, for years to come—need no upkeep!

ANOKA, Minn., has adopted a more efficient way to cut down accidents. Traffic is now kept safely apart by a permanent traffic cop —WHITE concrete islands!

This highly visible strip of white concrete is raised above the road surface. Motorists can see it day or night, in all kinds of weather. And, made with Atlas White portland cement, its whiteness is *permanent*. So it will dominate the road for years and years to come—without repainting, replacing, or other upkeep.

Think what this can mean to a city's budget —what it will mean to yours . . . when you put this road-divider to work, cutting down your accidents and your maintenance expenses!

White islands made with Atlas White, like the one above, can be placed on the job or pre-cast. Universal Atlas Cement Co. (United States Steel Corporation Subsidiary) Chrysler Bldg., N. Y.

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ATLAS WHITE PORTLAND CEMENT FOR MARKERS AND CURBS

ENGINEERING NEWS-RECORD

NEWS

Volume 123 AUGUST 17, 1939 Number 7

First Contract Let For Naval Air Bases

The first contract for new and enlarged naval air bases in the Pacific Ocean under the recently authorized naval construction program (*ENR* Apr. 27, 1939, p. 539) has been let by the Bureau of Yards & Docks, U. S. Navy, to a combination made up of the Hawaiian Dredging Co., Honolulu, Hawaii, Raymond Concrete Pile Co., New York, and the Turner Construction Co., New York. The contract is on a cost-plus-a-fixed-fee basis, the fee being $898,000 on work estimated to cost $14,600,000.

The work covered in the contract includes new bases or additions to existing facilities as follows: Kaneohe Bay, Hawaii, $5,714,000; Pearl Harbor, Hawaii, $2,633,000; Midway Islands, $3,720,000; Johnston Island, $1,030,000; and Palmyra Island, $1,510,000.

Congress has authorized the expenditure of $66,800,000 on air bases within the next three years. In order to speed the letting of contracts, Congress authorized the Bureau of Yards & Docks to award contracts to reputable and qualified contractors by negotiations rather than by public call for bids.

• • •

Merritt Parkway Passes Millionth Toll Car

Toll counters on the Merritt Parkway in Connecticut on Aug. 13 checked the millionth car since tolls were inaugurated on June 21, 1939. The average daily traffic during this period was 18,800 vehicles, or 4,700 vehicles per lane per day. The toll is 10¢ per vehicle.

On Aug. 7 the first fatal accident occurred on the 20-mi. parkway since its opening to traffic on June 29, 1938. A driver in the early hours of the morning fell asleep and his car ran off the roadway into a tree, killing one of the occupants. While a number of other accidents have occurred, most of them were light and none resulted in deaths. This represents one fatality in slightly less than four million vehicles, or one in 75 million vehicle miles.

Jurisdictional Disputes Ended By Contractor-Labor Agreement

Building and Construction Trades Department of American Federation of Labor and Associated General Contractors agree on first major cause of trouble in construction

Construction work upon which American Federation of Labor workmen are employed will no longer be disrupted by jurisdictional disputes, according to terms of an agreement reached last week by committees of the AFL and the Associated General Contractors of America meeting at Atlantic City last week. Under the agreement, unanimously approved, the labor executive council instructed all international union presidents immediately to return to work all men on jobs stopped because of jurisdictional disputes between trades affiliated with the department. All work now in dispute is to remain in possession of the trades in possession at time of stoppage and the work is to proceed pending a decision by the president of the department on the matter under dispute. Disputes arising from now on are to be referred to the president of the department without stoppage of the work.

Labor questions discussed

Conferences between the contractors and the union executives began on Tuesday and continued through Friday. They were the outgrowth of a resolution adopted at the March meeting of the AGC which authorized the managing director, Edward J. Harding, to

(Continued on page 38)

Wide World

WHERE 24 DIED IN WRECK OF STEAMLINER

When the "City of San Francisco," $2,000,000 streamlined train running between Chicago and San Francisco, was wrecked Aug. 12 in the Humboldt River canyon 40 miles west of Elko, Nev., 24 lives were lost. The train was traveling between 50 and 60 miles per hour and 13 of the 17 cars were derailed, six falling into the stream channel through the destroyed bridge. Railway officials give the cause of the wreck as sabotage, explaining that one rail was found to have been moved inward four inches and spiked.

Emergency Program Suffers Setbacks As Congress Favors Regular Works

Legislation passed by the last Congress summarized and the major appropriations for construction work listed

Domination of Congress, in its closing days, by a coalition of Republicans and conservative Democrats resulted in the 1940 fiscal year becoming the second in which no program of emergency public works is provided. The house refused even to consider a spending measure which in its original form included 100 per cent loans to non-federal public works and an unprecedented program of loans for major highway improvements. Inclusion of this later feature, however, along with legislation introduced in the closing days of the session, suggests an active effort next year, when a new quadrennial highway authorization comes up, to make important changes in the system of federal aid to highways.

Regular federal public works, such as rivers and harbors and reclamation, fared fairly well, however. An exception to this was the refusal of the House to consider authorizing USHA to loan an additional $800,000,000. A strong but unsuccessful effort was made by the House to restrict drastically the operations of TVA, and a similar House vs. Senate battle developed over the inclusion of Gilbertsville in the TVA appropriation. Important restrictions were imposed for the first time on the operations of WPA, and an end of session effort to modify these was unsuccessful.

Following is a list of the more important legislation acted upon by Congress:

Legislation passed

Relief: Restricting size of WPA building projects; dropping prevailing wage on WPA; firing WPA workers after 18 months; limiting WPA overhead expense; imposing other restrictions.

Reorganization: Permitting the President, with certain restrictions, to reorganize executive agencies of the government. He subsequently created the Federal Works Agency, embracing PWA, WPA, USHA, the former Public Buildings Branch of Procurement, and the former Bureau of Public Roads; he also created the Federal Loan Agency and the Federal Security Agency and shifted REA to the Department of Agriculture.

Panama Canal: Authorizing construction of a new set of locks at a cost of $277,000,000. Authorizing cost-plus contracts on military works in the Canal Zone and in Alaska.

Reclamation: Authorizing variation in annual repayments by water users according to variation in crop return. Authorizing postponement of 1939 payments in cases where water users are in distress.

Tennessee Valley Authority: Authorizing TVA to borrow money to purchase generating facilities of Tennessee Electric Power Co.; reducing TVA's borrowing capacity from $100,000,000 to $65,000,000. As passed by the House, bill put geographical limits on TVA operations and required Congressional authorization of all new power facilities, but these provisions were dropped in conference.

Housing: Reestablishing FHA-guaranteed improvement and repair loans until July 1, 1941.

Flood Control: Making the Muskingum system part of the Ohio Valley flood control project and authorizing reimbursement of the district for $1,500,000 spent for rights-of-way; permitting use of flood control appropriations for preliminary surveys, and authorizing certain preliminary surveys.

Reconstruction Finance: Continuing the RFC until July 1, 1941.

Highways: Permitting payment out of federal-aid allotments of half cost of toll bridges made free before July 1, 1941.

Housing: Provides that the 1940 census include figures on quality of housing.

Highway Officials: Expressing the appreciation of Congress to the American Association of State Highway Officials for its work in promoting road improvement.

Rio Grande River: Giving the approval of Congress to the Santa Fe interstate compact dividing the waters of the Rio Grande.

Bridges: Relieving bridge owners of all the costs of reconstructing bridges in the interests of navigation except those directly chargeable to benefits to the owners.

Legislation in conference

Inland Waterways: Making carriers on inland waterways subject to jurisdiction of the Interstate Commerce Commission.

Bonneville Power: Making the administration at Bonneville an agency of the Department of the Interior; creating the posts of assistant administrator, chief engineer, and chief counsel.

Legislation killed

Spend-Lend: Program of loans to self-liquidating projects passed Senate in mutilated form, but House refused to consider it. Original measure provided for loans for non-federal public works, highways, railroad equipment, rural electrification, farm tenancy loans, foreign loans. Before the House killed it the Senate had inserted reclamation loans and knocked out highway and railroad loans.

Housing: Authorizing USHA to loan an additional $800,000,000 and to make additional annual subsidies of $45,000,000—passed Senate but House refused to consider it.

(Continued on page 41)

APPROPRIATIONS APPROVED BY CONGRESS

Relief:	
Additional funds for fiscal year 1939	$825,000,000
Relief funds for fiscal year 1940	1,755,600,000
Civilian Conservation Corp (independent office)	295,000,000
Water and Hydro-electric projects:	
Rivers and harbors (War Department)	96,000,000
General flood control (War Department)	133,000,000
Mississippi flood control (War Department)	39,000,000
Fort Peck power installations (War Department)	2,000,000
Bonneville power installations (War Department)	7,000,000
Bonneville transmission lines (Interior Department)	13,000,000
Tennessee Valley Authority—including Gilbertsville	39,003,000
Bureau of Reclamation—from Reclamation Fund	11,383,000
Bureau of Reclamation—from general funds	45,700,000
Irrigation in the Great Plains (Bureau of Reclamation)	5,000,000
Indian land irrigation (Indian Service)	3,064,000
Highways:	
Primary road system (Public Roads Administration)	100,000,000
Secondary road system (Public Roads Administration)	15,000,000
Grade crossing elimination (Public Roads Administration)	20,000,000
Buildings:	
Buildings outside D. of C. (Public Buildings Administration)	30,000,000
Buildings in D. of C. (Public Buildings Administration)	15,800,000
Defense public works:	
Twelve navy air bases	63,000,000
Navy public works	55,000,000
Seacoast defense, including trans-Isthmian highway	8,000,000
Army air corps bases (this not specifically appropriated but included in War Department military appropriation)	62,000,000
Miscellaneous:	
Panama Canal—maintenance and operation	24,775,000
U. S. Geological Survey	3,293,000
Coast and Geodetic Survey	3,125,000

WASHINGTON HIGHLIGHTS

Special reports by Paul Wooton

FORTY-FIVE Justice Department investigators are working in 26 cities getting evidence to sustain the indictments that are to be brought in the near future alleging violation of the antitrust statutes in the handling of building materials. An oral announcement to the foregoing effect was made last week by Attorney General Frank Murphy. An outline of the program of the department was published in our issue of July 6, p. 20.

RECLAMATION interests are not disappointed by the failure of Congress to vote the $90,000,000 carried in the lending bill. They realize that other features of the bill were not popular in the West and that the $90,000,000 had been tacked on as a bid for western votes. Western interests realized that any self-liquidating highways would have to be built in the East. It also was feared that the appropriation for roads carried in the lending bill later might be made the excuse for reducing the regular federal aid program. Westerns also realized that little if any of the money intended for farm tenants would be loaned in the west. It was with the idea of balancing those items that the reclamation amendment was proposed when it was assumed that the bill would pass.

AS THE RESULT of the failure of the spend-lend bill to pass, PWA will start laying off men Sept. 11 and each pay day thereafter. It is expected that the present field staff of seven thousand and Washington staff of three thousand will be down to less than half its present strength by the year end. It is possible that some regional offices may close.

• • •

Central Park Subway Urged by N.Y. Board

A two-track subway to pass under New York's Central Park along the line of Seventh Ave. from 59th St. to 145th St. is proposed by the city's Board of Transportation in a report presented to the city's planning commission Aug. 10. The new line would be an extension of the Broadway line of the B.M.T. subways which turn eastward at Seventh Ave. and 59th St.

The projected subway is the major change in previous plans for expansion of the city's subway system as prepared by the Board of Transportation. The board's six-year program calls for a capital budget of $860,000,000.

• • •

Andes Railway Tunnel To Be Enlarged

The railway tunnel through the Andes Mountains connecting Argentina and Chile will be reconditioned to also permit highway traffic, according to the transportation division of the U. S. Department of Commerce.

Travel by highway over the Andes is ordinarily suspended for about seven months each year on account of heavy snowdrifts in the passes. Reconditioning of the tunnel is expected to make possible travel for nine months.

This new provisional highway will be used until a tunnel through the Andes is constructed as part of the Pan-American highway.

British Press Combine Photo

A NEW HOUSING PROJECT COMPLETED IN WALES

A RECENT modern housing project opened abroad is the White City Estate in Hammersmith, Wales, shown above. The estate provides 2,000 apartments in its 5-story buildings. A large part of the ground area is given over to courts.

Engineers to Make Nicaragua Survey

Proposed shallow draft barge canal to be studied by army engineers

A board of engineers from the army engineer corps started for Nicaragua August 10 to survey and study the route of a shallow draft barge canal across Nicaragua from the Atlantic to the Pacific. The survey is a result of conferences some months ago between President Roosevelt and President Somoza of Nicaragua at which it was agreed that such a canal, by opening up the interior, would improve trade relation between the two countries.

A defense value is also attributed to the proposed canal in that it would be possible in an emergency to move considerable quantities of material by barge, taking some of the load off the Panama Canal.

The eastern end of the canal would be formed by canalization of the San Juan River between the Atlantic and Lake Nicaragua. Location of the connection between the lake and the Pacific Ocean is one of the tasks before the board.

The entire task of the board is fourfold: it will locate and estimate the cost of the canal; it will estimate the cost of a highway paralleling the canal; it will estimate maintenance costs of both structures; and it will investigate terminal facilities for the canal.

The proposed canal would be for barges of 10 to 12 ft. draft rather than for ships. Surveys for the proposed ship canal were completed some time ago.

Head of the survey board is Lieut. Col. Charles P. Gross. Other members are: Lieut. Col. Paul R. Hawley, medical corps, and Capt. L. R. Groves; Capt. T. H. Stanley, Lieut. W. B. Bunker; E. E. Abbott, Cater Page, and E. F. Heyler, all of the U. S. Engineers. The first four were members of the board which made the studies for a Nicaraguan ship canal.

• • •

Landmark in Chicago To Be Torn Down

The Great Northern Hotel, Chicago landmark erected in the '90s and once the center of society and night life there, will be razed in the building of Chicago's new subway.

The structure is on a floating foundation and new foundation support would be needed. A cost of $150,000 to do this work is believed to be unwarranted by the owners.

Agreement Ends Labor Disputes

(Continued from page 35)

take action toward obtaining for general contractors working under agreements with organized labor conditions under which: (1) Uniform conditions covering daily and weekly hours of employment will prevail for all crafts engaged on the work; (2) Wage rates to be paid and working conditions to be observed shall be maintained at the level prevailing when the contract is signed, until the completion of the project; (3) Adequate provision is made for the training of apprentices in the skilled crafts in such proportion that the ranks of skilled workers be constantly repleted; and (4) Disputes or misunderstandings arising as to union craft or territorial jurisdiction be promptly settled giving due consideration to the interest of the employer through the most effective means without stoppage of work. Work shall proceed with the craft controlling the work previous to the dispute.

Under the authority of this resolution representatives of the AGC and AFL met to discuss labor problems. The agreement on jurisdictional disputes was the first outcome of discussions that are to continue in the future for consideration of other matters.

The contractors' committee headed by E. P. Palmer of New York included in its membership, H. A. Dick, Portland, Oregon, J. S. Miller, Pittsburgh, Pa., B. P. Larkin, Ridgefield, N. J., E. J. Dunnigan, Sr., St. Paul, Minn., B. F. Modglin, San Francisco, Calif., W. A. Klinger, Sioux City, Ia., A. C. Tozzer, New York, N. Y., W. S. Bellows, Houston, Texas, and George B. Walbridge, Detroit, Mich.

Speaking for the contractors, Mr. Harding declared the agreement is the signal for construction to go ahead. "We are mightily pleased with the action of the department," he said, "and we will continue to work with President Coyne and his associates to develop ways and means for advancing our mutual interests."

New department head

John Coyne, an engineer, was elected president of the Building and Construction Trades Department of the American Federation of Labor at the meeting of department officials held in Atlantic City last week. His election fills the vacancy caused by the death of Joseph A. McInerney. George Masteron, a plumber, was elected vice president and member of the executive council.

Friant Dam Bid Opening Postponed to Sept. 7

The date of opening bids for the construction of Friant Dam, originally scheduled for Aug. 23, has been postponed to Sept. 7, according to the Bureau of Reclamation. The postponement is due to a change in the specifications with respect to the method of placing concrete.

The dam is being built on the upper San Joaquin River (see *ENR* July 6, p. 35) and must be completed 1,200 days after the contractor receives notice to start the work.

Bids will be opened at Sacramento.

• • •

Trade Commission Charges Suppressed Competition

Conspiracy to suppress competition in the sale of vitrified clay sewer pipe has been charged in a Federal Trade Commission complaint against certain manufacturers and officials of that industry, all of whom are members of the Southern Vitrified Pipe Association which controls the output of pipe in the section of the country lying east of the Mississippi and south of the Ohio and Potomac rivers.

The complaint alleges that for five years the association and members have agreed among themselves on prices and conditions of sales of their products, including quotations made on a delivered basis only, freight rates from their respective shipping points being equalized to make prices identical at a given point.

It is also charged that by a united policy of refusing to quote government purchasing agents an f.o.b. mill price the manufacturers have deprived the federal government of the opportunity to benefit from land grant and other special railroad rates.

• • •

More Time Allowed For Bridge Study

Park officials of New York state have been granted more time by Major Philip G. Bruton, U. S. district army engineer, to study a change in location for the proposed $4,250,000 Rainbow bridge at Niagara Falls. A location is being considered whereby the bridge would be built on the site of the old "Honeymoon" span, destroyed in January, 1939.

Plans now being studied provide for a bridge 960 ft. long between abutments with a deck width of 58 ft. there being two 22-ft. roadways, a center curb 4 ft. wide and a sidewalk 10 ft. in width on the southerly side. Clearance above river would be 192 ft.

Before making his final report to the Secretary of War, Major Bruton will await the result of the conference between the Frontier Parks Commission and the Niagara Falls Bridge Commission.

Illustration from L'Ossature Metallique

FAMOUS BRIDGE CELEBRATES FIFTIETH YEAR

In 1889, when it was placed in service, the Landsdowne Bridge in India, pictured above, held the record for the longest span metal bridge in the world. Like the Forth Bridge in Scotland which, upon its completion a year later, assumed the span-length record, the Landsdowne Bridge is a cantilever structure, its two arms of 310 ft. and its suspended section of 200 ft. making its main span length 820 ft. Designed by Sir Alexander Rendel, the bridge required 3,000 tons of steel and cost something over one million dollars. Its fifty years of service as a railroad crossing of the Indus River is celebrated this year. The bridge is decked to carry highway traffic.

Muskingum District to Get Flood Control Costs

The federal government undertakes to reimburse the Muskingum Watershed Conservancy District for all its reasonable costs in acquiring rights of way and relocating structures for the Muskingum project in a bill which has passed Congress. The 1938 flood control act adopted the policy of federal payment of all costs of flood control projects, and made this retroactive to projects authorized in 1936, including the authorized Ohio River project.

Muskingum was not then a part of the Ohio project, so a special provision was incorporated authorizing reimbursement of the Muskingum district up to $4,500,000. This, it developed, did not cover the full costs incurred by the district. The present measure makes Muskingum part of the Ohio project, and authorizes immediate payment from flood control funds of a further $1,500,000 to the district.

Other minor changes made in the flood control law include an authorization to use regular flood control appropriations for preliminary surveys; previously a special $5,000,000 appropriation was made for surveys. The new law conforms to current practice.

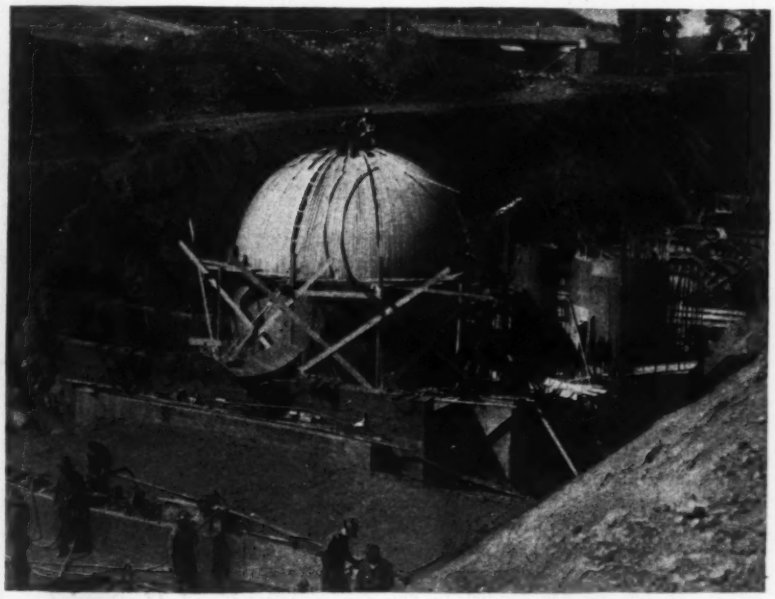
Port Authority Disbands Its Construction Staff

With all its construction operations now under way scheduled to be completed by March of next year and with no new major construction operations planned, the Port of New York Authority has begun to disband its large engineering force.

The design division will finish its work and will have been completely disbanded by Jan. 1, 1940. The construction division, greatly reduced, will continue until all work in the field is completed, which will be about Mar. 1, 1940. The inspection and testing division, which has been engaged on work for the Port Authority, the Triborough Bridge Authority and the New York Tunnel Authority, will be reduced but will be continued to perform service for the tunnel authority as long as required.

J. C. Evans, chief engineer, is being retained and will be placed in charge of all maintenance and mechanical and electrical operations. O. H. Ammann, director of engineering, will become consulting engineer.

A large number of men have been notified that their services will not be required after Oct. 1 and others after Dec. 31 of this year.



OUTLET WORKS OF VALLECITO DAM IN COLORADO

THE forms were nearly ready for the chamber which will house the 5 x 5 ft. outlet gates at Vallecito Dam on the Pine River in Colorado when the above photo was taken. The dam, an earth and rock structure being built 14 mi. upstream from Bayfield by the Bureau of Reclamation, will supply sufficient water for irrigating 30,000 acres and supplement the supply for another 36,000 acres already under development.

Drought Continues In Northeast

July rainfall offset by below normal precipitation during first two weeks of August.

Rainfalls slightly below normal through the first half of August have prevented the heavy rains of late July from bringing about more than a slight recovery of river stages in the northeast drought area. This was indicated by a telegraphic survey made by the U. S. Weather Bureau for *Engineering News-Record* early this week.

The Hudson River at typical gaging points is about a foot higher than the July lows. These, however, were 5 to 10 ft. above record low stages. The Connecticut River at Hartford is still at the July level—a stage about as low as has ever been reached except under special wind and tide conditions. Upstream, at Holyoke and White River, gains of about a foot have been made from July levels 3 or 4 ft. above record lows. The Merrimac has made only a slight recovery from stages within a foot of the all-time lows.

Most severe effects of the July drought were felt in the southern and Catskill areas of New York. Precipitation for the state as a whole, 2.42 in., was only 61 per cent of normal and was lower than any year of record (running back to 1890) except 1936 and 1933, when July precipitation was 2.13 and 2.14 in. respectively. Precipitation, moreover was very spotty, and in some parts of the state was the lowest in from 50 to 60 years. Stream flows in several cases were the lowest on record; Owego Creek, Roundout Creek, Chenango, and the upper Susquehanna all set new low records. Rainfall figures given include the heavy rains of the last few days of the month.

Pennsylvania, though dry, was not in a situation comparable with recent drought years. The 3.20 in. precipitation was 75 per cent of normal. In this state, however, May and June rainfall had been only about two thirds of normal, whereas in New York the months preceding July had been nearly normal. Flow of the Delaware and lower Susquehanna were at least twice the record lows of other years.

New England precipitation during July averaged 2.32 in., 62 per cent of normal and lower than any except the 1929, 1898, and 1888 rainfalls of 2.18, 2.17, and 2.12 in. Effects of the drought were most severely felt in the southern states of New England. Stream flows throughout the area, although about half normal for the month, were well above previously established minima.

OBITUARY

MICHAEL CREEDON, 85, retired building contractor and for ten years highway commissioner of Dobbs Ferry, N. Y., died Aug. 4 in New York City.

JAMES E. CURTIS, 56, construction and civil engineer, died Aug. 4 in Washington, D. C. He received his engineering education at George Washington University and soon afterwards was employed by the District of Columbia on waterworks design and building construction. For nine years he was assistant superintendent in direct charge of maintenance and operation of Washington's water supply and purification work. In 1921 he was superintendent in charge of construction of the Dalecarlia filter plant. Becoming senior engineer in 1929, he supervised the entire water supply system of the district from 1929 until his death.

WADE H. APPLE, building contractor, died in Greensboro, N. C., Aug. 10, at the age of 62.

FRANK T. HOWELL, 83-year-old retired contractor, died in Asheville, N. C., Aug. 7. He was engaged in the general contracting business in that city for many years and from 1923 to 1927 served as city building inspector, retiring about six years ago.

ADAM L. BECK, 77, president of the Great Lakes Portland Cement Co., of Buffalo, died Aug. 10. He received his education at Northwestern College and entered the construction field at the age of 21, building roads, bridges, streets, and other public works in Indiana. He soon became secretary and director of the Huntington White Lime Co., and had served as president of the Mitchell Lime Co., the Oklahoma Portland Cement Co., and the Indiana Portland Cement Co. In 1925 he organized the Great Lakes company in Buffalo.

WILLIAM T. KEEFE, 38, senior chemist of the Portland laboratory of the Connecticut State Highway Department, died Aug. 9 at Bridgeport. He was a graduate of Worcester Polytechnic Institute and a member of the Connecticut Society of Professional Engineers.

GEORGE J. TEPE, 57, senior engineering aid of the Department of Works of Louisville, Ky., died Aug. 10.

CHARLES A. ROWLAND, building contractor at Louisville, Ky., for 20 years, died Aug. 11 at the age of 59.

LEO C. DZIATZKO, 62, civil engineer, died Aug. 11 in Webster Grove, Mo. He graduated from Washington University in 1895 and until two years ago had maintained an office as a consulting civil engineer, joining the Home Owners Loan Corp. at that time.

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Healy Is Low Bidder On Chicago Subway

The Healy Subway Construction Corp. was low bidder Aug. 10 when bids were opened for twin shield driven tunnels 4,500 ft. long to be constructed under Dearborn St. in Chicago. The company's bid was $6,436,855, and the Griffiths Subway, Inc., was second low bidder, bidding $7,133,000.

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Army & Reclamation Bureau to Divide Dam Work

Efforts are now underway to work out a procedure for allocation of dam building work between the Bureau of Reclamation and Army Engineer Corps. Flood control, a relatively new activity of the federal government, has been assigned to the army engineers. But most up-river reclamation projects have flood control elements, and vice versa. So there is plenty of room for overlapping jurisdiction. This is especially true when it is remembered that the bureau sometimes functions as a dam-building agency; it is now building Marshall Ford Dam, and it designed Madden Dam at Panama and Norris Dam on the Tennessee—none of them reclamation projects.

It has frequently happened that the corps and the bureau make reports on the same project, and in the past no satisfactory method has been worked out for avoiding such duplication. Recently both agencies have made studies of the Pine Flats and Kings River projects in California and the Wagon Wheel Gap and Conejos projects in Colorado and New Mexico. None of these reports have been released as yet, and it may probably be assumed that one reason is divergence between findings of the two agencies; historically the army has tended to allocate larger proportions of cost to flood control and navigation, while the bureau has tended to emphasize reimbursable elements such as reclamation and power.

A further difficulty has arisen when land-owners have sought to buy reclamation water from straight flood control projects. Since these projects are so drawn up that reclamation water is a byproduct, the bureau has found it hard to arrange contracts incorporating payments comparable with those made by water-users on other projects. A case in point is Conchas Dam in New Mexico, where a conservancy district was formed after the project was well underway.

Coordination plans

The President recently conferred on these matters with the Gen. Schley, Chief of Engineers, and Commissioner of Reclamation Page. He said afterwards that although the question had been under discussion since the first studies of reorganization, there was no intention of forming a central dam-building agency. In general, he said, work in which navigation and flood control are the major elements would be assigned to the Engineer Corps, while work in the non-navigable upper rivers would ordinarily go to Reclamation. A further factor would be the amount of work which each agency had on hand at the time. Reports made by either agency would be referred to the other for study.

The outlines of a procedure for handling multi-purpose projects are included in the recently-passed repayment bill, in which it is provided that the Secretary of the Interior make allocations of cost as between reclamation, power, and municipal water supply, and consult with the Chief of Engineers on the allocation to flood control and navigation.

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Workers' Memorial

A MEMORIAL plaque to eight workmen killed in a cofferdam disaster during the construction of the Port Stanley, Ont., bridge was unveiled at the bridge opening ceremonies recently by King George VI. The bridge is a double-leaf bascule bridge across the harbor of Port Stanley.

British Engineers to Study American Highway Practices

Aiming to stimulate interest in up-to-date roads suitable for the increasing traffic in their country, a party of English engineers, Parliament members, county councillors, county surveyors, chief constables, and municipal authorities will come to the United States in September.

A number of tours will be made in the eastern United States to visit various engineering works, including express highways on Long Island, Lincoln Tunnel and the George Washington Bridge at New York, the Pulaski skyway in New Jersey, and the naval testing station at Washington, D. C.

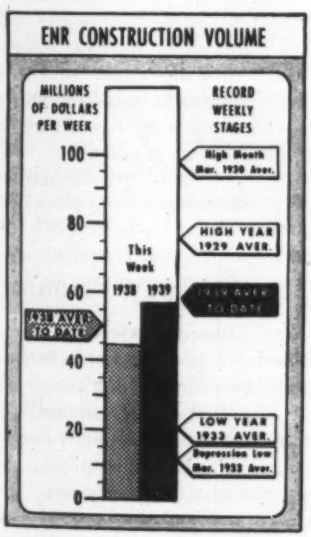
CONTRACTS AND CAPITAL

ENGINEERING construction awards for the week, $60,851,000, are 37 per cent above the corresponding week in 1938, but are 18 per cent below the high volume of a week ago. This is the second successive week that 1939 awards have topped their respective 1938 values.

The construction total for 1939 to date, $1,921,456,000, is 16 per cent higher than in the initial 33-week period a year ago.

Private construction for the week is the fourth highest of the year, tops the 1938 week by 176 per cent, and is 42 per cent above the preceding week. The high industrial building volume is responsible. Public construction is 6 per cent above a year ago, but 34 per cent under last week.

Industrial buildings reached their highest weekly level since June 24, 1937, due primarily to a $10,000,000 oil refinery project in Illinois, and a $3,000,000 paper plant in Texas. In addition to industrial buildings, waterworks, sewerage, public buildings, earthwork and drainage, and unclassified construction exceed their respective totals for the 1938 week, and waterworks and sewerage top their last week's totals.



New capital for construction purposes for the week totals $311,744,000, a 463 per cent increase over the volume for the 1938 week. This week's financing is made up of $296,000,000 in federal funds for WPA construction, 25 per cent of the total estimated WPA construction appropriation; $11,073,000 in corporate security issues; and $4,671,000 in state and municipal bonds.

New construction financing for 1939 to date, $2,122,480,000, is 21 per cent lower than in the 33-week period last year.

CONTRACTS

(Thousands of dollars)

	Week Ending Aug. 18 1938	Aug. 10 1939	Aug. 17 1939
Federal	$7,607	$18,474	$6,719
State & Municipal	28,755	40,235	31,782
Total public	$36,362	$58,709	$38,501
Total private	8,117	15,725	22,350
TOTALS	$44,479	$74,434	$60,851
Cumulative 1939 (33 weeks)			$1,921,456
1938 (33 weeks)			$1,652,128

Note: Minimum size projects included are: Waterworks and waterways projects, $15,000; other public works, $25,000; industrial buildings, $40,000; other buildings, $150,000.

NEW PRODUCTIVE CAPITAL

	Cumulative 1938 33 Wk.	1939 33 Wk.
NON-FEDERAL	$1,548,670	$994,452
Corporate securities	255,959	152,799
State & Mun. bonds	368,945	397,501
R.E.A. loans	11,629	89,445
R.F.C. loans	96,891	3,855
P.W.A. allotments	459,218	.....
U.S.H.A. loans	154,528	160,852
Fed. Aid—Highways	201,500	190,000
FEDERAL	1,143,942	1,128,028
TOTAL CAPITAL	$2,692,612	$2,122,480

Fed., '39 total includes 25% of $1,182,000,000 estimated for 1939-40 WPA construction.

FHA MORTGAGES

	Week Ending Aug. 13 1938	Aug. 5 1939	Aug. 12 1939
Selected for appraisal	$23,372	$19,149	$20,500*
Cumulative 1939 (32 weeks)			$720,504
1938 (33 weeks)			$601,647

*Subject to revision.

ENR INDEX NUMBERS

Index Base		1913	1926
Construction Cost	Aug.	234.90	112.92
Building Cost	Aug.	196.54	106.24
Volume	July	166	73

BRIEF NEWS

THE 70,000-acre irrigation project being built in Willacy county, Tex., will be placed in operation about Sept. 15, according to George M. Bull, PWA regional director. The work cost $5,000,000. Water is being taken from the Rio Grande River.

THE MISSOURI LEGISLATURE in a recent session provided for an increase in the personnel of the state highway patrol by 50 men. The addition will bring the patrol strength to 165, to date 6,500 applications having been received for the vacancies. Many members of the patrol are college graduates.

DURING the first three months of 1939 a 2 per cent tax was collected on nearly $1,000,000 worth of building material used in Iowa, according to state tax commission figures. The building material yielded 28 per cent of total tax revenue collected.

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Legislation Record

(Continued from page 36)

Panama Canal: Authorizing additional annuities to men who worked on construction of Panama Canal—vetoed by President.

Housing: Would raise cost limits on USHA projects in metropolitan areas of 500,000 population to the level permitted in cities over 500,000. Passed Senate but was attached in House committee as amendment to the new USHA authorization and died with it.

Legislation pending

Stream Pollution: (S685—passed Senate) Setting up a division of water pollution control in the Public Health Service and providing for loans to aid pollution elimination; this bill was passed last year but vetoed for technical reasons.

Rivers and Harbors: (HR 6264—passed House) Authorizes $83,000,000 of river and harbor projects; Senate committee increased to $407,000,000, but Senate did not act on bill.

Highways: (S 25—passed Senate) Prohibits interstate operation of motor vehicles except by drivers licensed by a state whose license requirements meet certain standards.

(S 231—passed Senate) Authorizes Public Roads Administration to acquire and turn over to states forest lands along federal-aid highways in order to protect natural beauties.

Contracts: (S 835—passed Senate) Establishes workmen's compensation on federal projects in states not having a workmen's compensation law.

Engineers: (S 1128—passed Senate) Establishes licensing of engineers in the District of Columbia.

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NEWS CONTINUED ON PAGE 88

COMMENT AND DISCUSSION

Readers' opinions on matters that concern the engineer

Just Sightseeing

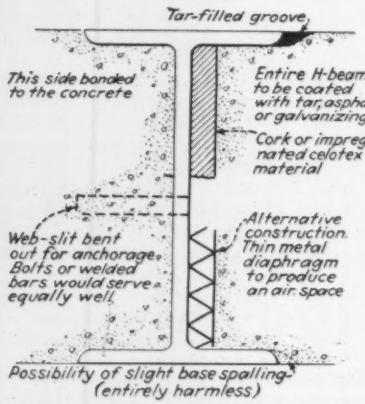
Sir: In the "Men at Work" page of your July 13 issue you show my picture in connection with the Deer Isle-Sedgwick Bridge. The caption gives the impression I was resident engineer on the construction of this bridge. This was not the case. The resident engineer on that job was, and still is, D. G. Letourneau.

R. Boblow
Robinson & Steinman
New York, N. Y.

Supplementary Ruminations On Pavement Joints

Sir: In whittling down my "Ruminations on Pavement Joints" (*ENR*, July 20, 1939, p. 77) to occupy *exactly* one page there was, I feel, some over-deletion, a portion of which should, in the interest of completeness, be restored. Moreover, in the light of the added thought and personal discussion that have developed since the article was written, there are two modifying aspects that should be injected into the picture, for the benefit of anyone who might contemplate giving this joint a practical workout. Omitted items and supplementary comments follow:

The sturdy one-piece construction, pre-curved to the exact surface-of-pavement cross-section, makes the joint-beam a construction asset because it can be firmly anchored at the curbs and used both as a finishing guide and a bridge to which vibrators and finishing tools can be attached if desired.


Tar-filled groove
This side bonded to the concrete
Entire H-beam to be coated with tar, asphalt, or galvanizing
Cork or impregnated celotex material
Alternative construction. Thin metal diaphragm to produce an air space
Web-slit bent out for anchorage. Bolts or welded bars would serve equally well
Possibility of slight base spalling (entirely harmless)

Modification of the pavement joint proposed and illustrated as Fig. 2 in the article.

The joint can be used with thickened-edge pavement construction; the depth of beam employed being either the center depth (letting the lateral edges of the slabs extend below the joint), or the edge depth which would produce thickened-edge transverse joints resulting in a very stiff and high-grade construction.

Used with short deformed bar tie-dowels at intervals through the web or with slitted web strips bent alternately into the slab on either side, and without expansion provision, the beam-type of joint should be ideal for the center or longitudinal joint performing, as such, the triple function of: load transfer, hinge, and pavement center line marker.

Further thought and discussion indicate that probably the alternative designs of Fig. 2 of the article (as reproduced here) are the only ones that should be considered, i.e.: (a) The joint beam should be of the H-beam type (parallel flange faces), rather than the I-beam type in order that there be no slope to the flange to create looseness of contact when slab is contracted (either cold or dry) or wedging when the slab expands from heat or moisture. (b) The air space or compressible filler should definitely be on but one side of the web, the other side being anchored firmly to the concrete. The simplest anchorage would probably be by longitudinal web slits bent out at right angles into the concrete, although bolts or welded lugs could easily be used. To have a beam floating between the two layers of compressible material might promote vibration and looseness under the pounding of traffic in spite of snug-fitting flanges top and bottom.

Fig. 1B of the article is a bit misleading, as drawn, since much of the flexing of a dowel bar doubtless occurs within the slab, the holes at entries of dowels being enlarged somewhat from the high local compression between the concrete and the dowel bar where the bar enters the concrete. In Fig. 2 of the article there should obviously be no bead of concrete between the edge of the top flange and the tar-filled groove; it has been removed in the reproduction herewith.

H. J. Gilkey
Head, Department of Theoretical and Applied Mechanics
Iowa State College, Ames, Iowa

Length of Parabolic Arc

Sir: The approximate formula which Paul E. Magerstadt uses on page 67 of your issue of July 20, 1939, can be improved by taking in another term of the series from which it comes, and by correcting a printer's error in sign, to read for a full horizontal span L

$$S = L + \frac{8}{3}\frac{k^2}{L} - \frac{32}{5}\frac{k^4}{L^3}$$

By substituting $2L$ for L the corresponding approximation in the different notation used by Messrs. E. I. Fiesenheiser and F. J. Sauer for their case becomes

$$S = L + \frac{2}{3}\frac{k^2}{L} - \frac{2}{5}\frac{k^4}{L^3}$$

Numerical values for the arc length, measured from the vertex, for the somewhat extreme case of a rise of 400 ft. in a half-span of 2,000 ft., are then

First approximation, two terms	2053.333 ft.
Second approximation, three terms	2052.053 ft.
Exact formula	2052.121 ft.

The difference of 0.068 ft. between the last pair of values seems quite insignificant in practice, because it is almost impossible to measure. Hence there seems to be no case where the trouble of using either of the long exact formulas is justified, and particularly so if only five figure logarithms are to be employed.

J. B. Macphail
The Shawinigan Engineering Co. Ltd.
Montreal, Canada

Little Highway Planning

Sir: Although it cannot be considered a typical example to be followed, the Merritt Parkway (*ENR*, July 20, 1939, p. 46) should help to bring to the minds of many engineers the crying need for long range planning in highway development.

For a moment, look at some states

which most of us can think of without much effort. On the surface, can it be said that planning of any kind exists? It would seem that highways over some of the states had been laid out with no thought of the future—in a very haphazard manner, so to speak.

Long range planning means many things, some of which are: possible development of existing highway into a super-highway similar to the Merritt Parkway; locating a new highway and grade eliminations at such points that may not materially hinder the future development of a city's suburban area; cooperation of city, county, state and federal highway engineers in the development of locations.

When every state has a planned highway procedure free from politics or the influence of politicians of every caliber, one will be able to say then, in truth, that highway planning is beginning to function.

REX L. FOSSNIGHT
Civil Engineer
Kansas City, Mo.

Continuous Beam Design

Sir: It has long been the desire of structural engineers to find a short and reasonably accurate method of designing continuous beams. I believe the method outlined below, which is a modification of the Theorem of Three Moments, has both these features.

Handbooks on reinforced concrete tell us that a beam with fixed ends will develop a negative moment of $0.6\ wl^2/8$ even if the beam is reinforced at the middle of the span for a positive moment of $wl^2/8$. They also tell us that the addition of steel over the support will relieve the positive moment. It follows that, if a continuous beam were designed for a positive moment of $wl^2/8$ and six-tenths of the steel bent up over the supports, the positive moment would be less than $wl^2/8$. Therefore, if the bottom portion of the beam were sufficiently strong in compression at the supports, we could add top steel over the supports until there would be very little positive moment at the middle of the span, in which case the beam would become, in effect, two cantilevers, each one-half the span in length. Although this is not a desirable condition, it does show that we may assume any negative moment (so long as it is equal to, or greater than, $0.6\ wl^2/8$) and modify the static moment accordingly.

Since shear in continuous beams varies as the negative moment, and since the positive moment varies as the shear, it seems logical that, if we assume negative moments equal to $0.6\ wl^2/8$ and then proceed to find the shears and positive moments as we would if we had used the Theorem of Three Moments, the analysis would be reasonably correct and considerably more economical than the customary method of using coefficients of 1/10, 1/12, etc.

A comparison of the moment curves obtained by using each of the three methods shows that the values obtained when the negative moments are made equal to $0.6\ wl^2/8$ fall between those figured by the Theorem of Three Moments and those figured by the standard coefficients. In the case of unequal spans the negative moment should be six-tenths of the static moment of the greater span.

Some building laws require beams and girders to be figured for a moment of $wl^2/8$. But, to obtain a safe structure, the beam or girder should have over the supports about six-tenths of the amount of steel required at the middle of the span. Obviously it is a waste of material to provide for a moment of $wl^2/8$ at the center of the span and also a moment of $0.6\ wl^2/8$ over the supports.

I. E. MORRIS
Birmingham, Ala.

Beauty in Design

Sir: To "reminisce" is a pleasant or a wearisome proceeding, depending on whether one is the speaker or the victim, but it often casts an interesting lighthouse gleam over the currents of life.

This pearl-of-wisdom remark springs partly from recent paragraphs in *ENR* about beautifying highways, but more particularly from an item some time ago showing a remote and unknown bridge pier on a western railroad which the designer had dared to make attractive. Such an idea, a generation or two ago, would not only have been roundly damned as nonsensical but would also have been full of danger, for any chief engineer who had countenanced it would almost certainly have found himself out upon the sidewalk, with a bruised feeling in the portion where kicks are usually bestowed. Utility was the god before whom we had to bow down; like Mr. Ford's first cars, made only to move about, our designing was limited to carrying out basic principles and nothing else.

Wasn't it Buddha who said that every created thing to perfect its destiny must combine the principles of beauty, truth and use? Or maybe it was John Ruskin. Well, anyway, somebody said it, and it is quite true, as much in engineering as in the less creative walks of life. If a structure does not embody truth, it will go haywire; if it won't be useful, no money will be found to build it; and if it is ugly, everyone who sees it will experience a pain, perhaps in the neck, perhaps elsewhere. Charles Macdonald, president of the long-gone Union Bridge Co., in its time equalling in prestige the American Bridge Co. of today, once said that he believed a bridge which would fulfill the function of carrying its load and yet possess artistic lines would be found to be the most economical in cost. Economy was then the sole desideratum, and art was believed to be its enemy. Thus it was an heretical remark for those days. It was many a long year before other engineers began to entertain the same idea and to apply it cautiously so as not to alarm the financiers. But a few successful examples won the public, and now, like the female voter whose first and last choice is the candidate who is the best-looking, the public seems to be primarily impressed by the appearance of an engineering structure, and its success comes second.

To my mind, one of the most remarkable and encouraging trends of this generation is the advent in all walks of life of a craving for beauty. Efforts to obtain it in engineering structures may be ranked as one of the major advances of the profession, for so many of these newer structures, whether they be roads or bridges or something else, are always in view, and so cannot fail to have a lasting influence upon the public. As the Wise King, himself one of the ablest executives in history, said more than two thousand years ago: "Where there is no vision, the people perish." And, per contra, where there is vision, the people live.

DAVID A. WATT
St. Petersburg, Fla.

ENGINEERING NEWS-RECORD

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Editorial Offices at 330 West 42nd Street, New York

AUGUST 17, 1939

JURISDICTIONAL DISPUTES have been the chief source of delay and increased cost in the construction industry for many years past. Last week the Building and Construction Trades Department of the American Federation agreed with the Associated General Contractors on methods for eliminating such disputes and at the same time, as evidence of good faith, ordered the ending of all strikes that now center on jurisdictional disputes. Hereafter all questions of jurisdiction are to be decided by a referee without stoppage of work. The present agreement, outgrowth of a movement that began at the A.G.C. meeting in San Antonio two years ago, is cause for congratulations as an outstanding development. It is true that earlier agreements for adjustment of jurisdictional disputes failed, but present conditions are more favorable to success. And jurisdictional peace is timely when the Department of Justice is beginning a study of high construction costs and when the C.I.O. is challenging the A.F.L. dominance of the building trades.

Something of a Record

AN UNUSUAL safety performance has been made by the Merritt Parkway, Connecticut's 20-mi. express highway. A week ago, after more than 13 months' operation, it reported its first fatal accident. High speeds prevail on the parkway, as it is posted for 50 mi. per hr., and the traffic is far from light, having averaged nearly 20,000 vehicles per day during the last two months, or 5,000 per lane, with maxima of nearly 3,000 vehicles per hour for the four lanes. These are conditions that usually lead to a substantial accident rate; yet prior to the recent fatal accident (a driver fell asleep and ran off the roadway into a tree) only ordinary light accidents had occurred. We believe this is the first high-speed trunk road of heavy traffic that has made so good a safety record. Evidently the safety elements built into the Merritt Parkway are effective: well separated roadways, good alignment and sight distance, ample widths and safe curbs.

Knowledge Evolves

IT IS NATURAL to think of the phenomenal growth in dimensions of bridges during the past few decades as the product mainly of increased size of problems and increased boldness in construction. However, L. S. Moisseiff's keen analysis of suspension bridge progress shows that this is not so, and that a long sequence of individual steps in the evolution of bridge design through analytical study and observation of bridge behavior forms the real background of the progression from Niagara and Brooklyn to George Washington and San Francisco. Because of this evolution each new major bridge undertaking gives further opportunity to develop suspension bridge design and in turn presents conditions which make it possible to extend our knowledge further. Whitestone and Tacoma are striking examples of the process.

Panama and Nicaragua

DECISION TO START WORK in the near future on a third set of locks at Panama Canal, reached by Congress in its closing hours, is welcome as an antidote to the renewed agitation for a canal across Nicaragua. Yet while Congress was considering this projected work at Panama the President was naming a board which has now sailed for Nicaragua to study the possibilities of canalizing the San Juan River for moderate-draft vessels and digging a canal from Lake Nicaragua to the Pacific, presumably also for navigation by small vessels. It is difficult to discover on what grounds this country could justify the expenditure of many millions on a canal other than one for our largest battleships, yet a study of such a canal and its costs was made only recently. It is to be hoped, therefore, that sending the present commission to Nicaragua means only that the President has loaned some of our experts for a study of what is essentially a local question.

Repairing Concrete Dams

MAINTAINING engineering structures and particularly concrete dams is preeminently a problem of the individual case, one that depends in high degree on the character of the structure

and the conditions of service and attack. Nevertheless such an operation as the refacing of Stevenson Dam offers a useful contribution to maintenance experience. Any form of facing for a dam is apt to require provision for the action of seepage water. Experience at Tunnel and Stevenson dams should show whether single-level drainage is likely to be adequate, and under what conditions a more complete system of distributed drains is required. In addition the Stevenson work brings to mind the problem of assuring inherent durability of concrete. That the material can be made durable even under the severe attack to which the overfall face of a dam is subjected may be concluded from the presence of areas of uneroded original surface. It should be possible to find how these differ from other areas, and to adopt construction practices that will produce durable material at all points.

Construction Looks Up

DEFEAT of the big lending bill in the closing days of Congress is a source of satisfaction, as evidence of a turn toward sounder legislative thinking. The assemblage of strange devices for passing out public moneys under the seductive name of self-liquidating investment could not inspire confidence; the shelving of the bill was therefore a strongly reassuring act.

In view, however, of the direct aid to construction contemplated in the provisions for loans to local public works and toll roads, more specific examination of the bill is in order. Its effect on construction is important because of the key position of construction recovery in the improvement of industry and business. There were many who believed that the bill by continuing PWA assistance to state and city public works would have brought about more building and employment.

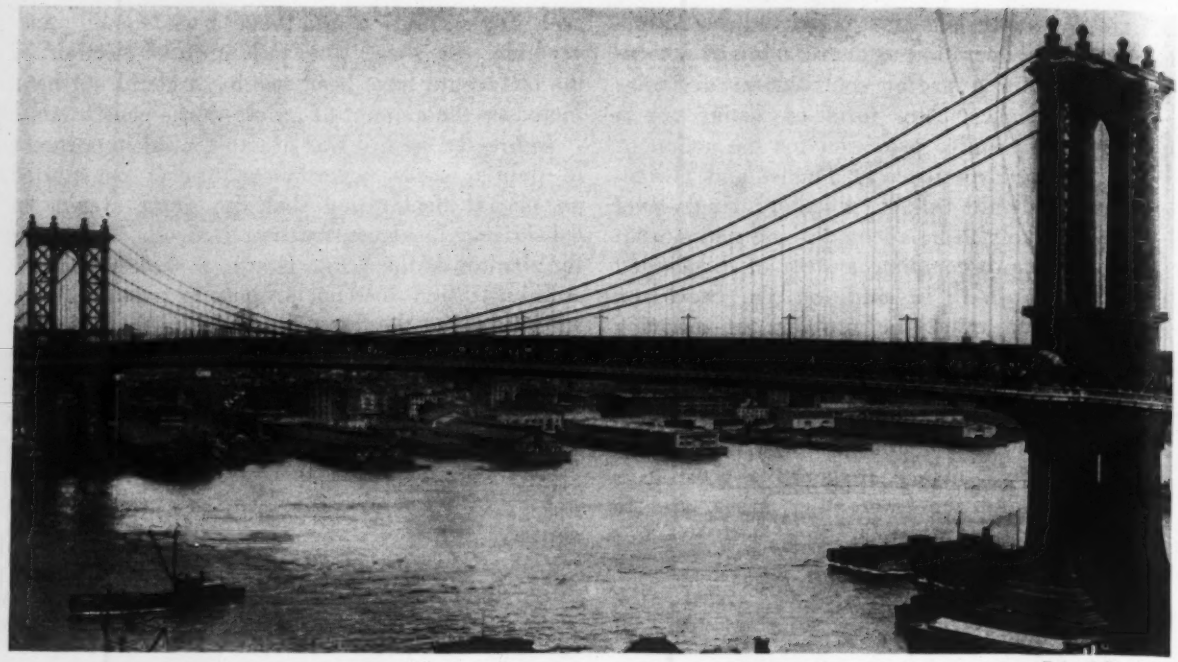
Study of the bill shows at once that the anticipated effect would not have been realized. The bill provided for loans only, without grants, and federal loans are and for several years have been of no interest to communities in good financial standing. Ample money is available at low rates, both for municipal bonds and for revenue bonds—the latter fact being demonstrated only recently by the advantageous sale of the Bay Bridge securities. On the other hand communities whose credit is impaired, or those proposing revenue-bond projects of doubtful earning power, would have applied for the federal loans but the record indicates that such applications would have been found unacceptable. In short, the public-works section of the bill would have been sterile. It could not have increased the amount of public-works construction.

Indirectly the bill was of substantial importance to public works nevertheless, for it constituted an official declaration that the grant system of subsidizing local construction is dead. Written in the shadow of the White House, it showed that the Administration does not propose to continue these subsidies, and the Senate's action on the bill gave evidence of a like attitude. This clearly spells the final end of public-works grants, particularly in view of the steady drift of public and Congressional opinion toward economy.

The significance of this development is plain, in view of the persistent stagnation of public-works activity. Current statistics reveal that during the period of federal bounty the states and cities reduced their works expenditures by more than the amount contributed by Washington, and that the net result of federal assistance was to decrease the volume of public construction. Cities by the score discontinued building public works other than through federal assistance; planning and self- reliant prosecution of city improvements went out of fashion. Now, with the ending of PWA grants, and with WPA operations rigidly restricted by the new relief law, communities no longer will be tempted to wait for gift money. They are bound to resume progressive modernization of their plant. Thus public-works construction should in due course rise to its normal dimensions.

Toll roads, the other major construction loan item of the lending bill, are too speculative to claim serious consideration in appraising the measure and its defeat. It is doubtful whether any toll-road projects have been fully studied, and still more doubtful whether a single one would justify a loan on the prospects of its toll earnings. It would be little less than fantastic to assume that toll-road loans would have revived construction—or that their effect on general construction initiative would have been other than harmful.

In summary, the bill contained nothing that could serve as a legitimate stimulus to construction. On the contrary its passage would have set back construction progress by giving rise to confusion and unsettlement. Such positive value as the bill possessed lay in the abolition of public-works subsidies, and this value remains. For these reasons the defeat of the bill is highly favorable to the progress of both public and private construction. In fact it may prove to be the starting point of a new and vigorous construction recovery.



Ewing Galloway Photo

FIG. I. MANHATTAN BRIDGE, NEW YORK, WHICH INTRODUCED CONCEPTS BASIC TO MODERN DESIGN.

Growth in Suspension Bridge Knowledge

LEON S. MOISSEIFF
Consulting Engineer, New York, N. Y.

***Contents in Brief**—Bridges that have contributed notably to our present state of knowledge are the Manhattan—first application of the deflection theory and first use of fixed cable saddles and fixed towers; the Philadelphia-Camden—first completely rational tower design; the George Washington—first recognition of extreme stiffness of the cables themselves; and Whitestone—first use of shallow plate girders as the stiffening members for long suspension spans.*

MODERN suspension bridges had their beginning in the first decade of our century, when the Manhattan Bridge over the East River in New York City was designed and built. This bridge contains many of the seeds which blossomed forth later in the suspension bridges of our present day; also it carries some survivals of past faults.

The main reason why the Manhattan Bridge is so important to a discussion of the development of modern suspension bridges is that it was the first such bridge to be proportioned and designed on the basis of the Deflection Theory. In other words, it was in the study of that bridge that it was clearly recognized that the deadload of a suspension bridge moderates to a very substantial degree the distortions of the structure under live load; furthermore, this knowledge was practically applied to the design of its parts.

Shallower stiffening trusses

The immediate consequence of this advance in the knowledge of the behavior of suspension bridges was the realization that trusses of great stiffness reduce but little the deflections of the bridge. Contrary to the behavior of ordinary girders and trusses, the vertical deflection of the bridge is not proportional to the moment of inertia of the truss; rather, it is dependent on the weight of the structure and its main proportions. It then logically follows that the depth of the trusses could be reduced without appreciably impairing the stiffness of the bridge. A great saving in the material of the trusses is realized thereby. The saving spreads from the trusses to the cables and towers that support the weight of this material and to the anchorages that fasten the cables to the ground.

The Deflection Theory has since attracted the attention of many engineers who have studied various approaches to it. At the same time the actual practice of designing suspension bridges has developed procedures for correcting its limitations and extending its applications. It has also received experimental verification on several large size models, the observations on which were conducted on lines of methodical research and with scientific precision.

The knowledge of the behavior of

suspension bridges gained from the basic principles of the Deflection Theory and the economy resulting from it represent the outstanding engineering achievement in suspension bridges in recent times.

Fixed towers designed for bending

Another fertile advance in suspension bridges was made in the design of the towers of the Manhattan Bridge by immovably attaching the cable saddles to the tops of the towers and the tower bases to the piers and so compelling the towers to follow the cable movements by bending. This advance was not the result of theoretical considerations, but of practical observations on existing strucutures. It had been observed on the Brooklyn and Williamsburgh bridges that the saddles, which are carried on rollers, did not respond to the daily variations in cable pull caused by live load and temperature and that they yielded, more or less suddenly, after an accumulation of unbalanced pull would overcome their friction. Such a condition produces bending in the towers, which is relieved intermittently by jerks and which does not promptly bring back the saddles to their due position, if at all. This generally results in eccentric reactions on the towers, causing stress the same as bending. By fastening the saddles to the towers and causing them to bend, an elastic action is produced that is continuous and uniform.

The practice of designing the towers to follow the saddle movements has eliminated the consideration of wide massive towers in which high bending stresses would be produced, and has led to the adoption of slender steel shafts. This was at first a revolutionary step. The towers of the Manhattan Bridge are only 290 ft. tall above the piers and have, at mid-height, a width of 16 ft., or 1 in 18, but so daring was the appearance of the tower at that time that for many years the author was haunted by its excessive slenderness; today they appear to him rather stubby.

It has been found in the study of later long span bridges that the correct consideration of the displacement of the towers and the forces acting on them will result in a design of the shafts which will require but little, if any, additional material to take care of the bending stresses This is due to the fact that the application of the common formula to the towers is correct for the vertical reactions, but it would be incorrect to add to it the bending stresses produced by the displacement of the tops. For such a condition as presented by a suspension bridge tower the actual curve of flexure of the tower should be established, and the stresses due to this position computed. The tower should then be designed for either the vertical or the deflected position. It is good design so to proportion the dimensions of the tower as not to have to add metal for the bending stresses. To the author's knowledge, this procedure was first applied to the Delaware Bridge at Philadelphia, which was built in 1921–26 with a span of 1,750 ft. Some time later the Ambassador Bridge at Detroit, with a span of 1,850 ft., was built with towers of 363-ft. height and a slenderness ratio of 60.

It was found desirable to build slender tower shafts because they must follow the pull of the cables due to the variations of live load and temperature, and the bending stresses produced by their displacements are proportional to the width of the shaft. It is also desirable to take care of the horizontal shear efficiently and economically. These considerations led to the closed type of construction for the shafts and, for the longer and heavier bridges, to the cellular type of tower, as exemplified in most recent long-span suspension bridges.

The principle of attached saddles and fixed tower bases has been followed in all long-span suspension bridges of recent times in the United States. For bridges having low towers it may become advisable to rest the towers on hinges.

Cable stiffness recognized

An immense step forward was made when the George Washington Bridge was built with a span of 3,500 ft., exactly double that of the Delaware Bridge. As befitted a structure of such unprecedented magnitude, it was accompanied by many studies of various details and by most important tests on column strength and strain observations on the actual structure, which have been described fully in engineering literature. The enormous dead load of the George

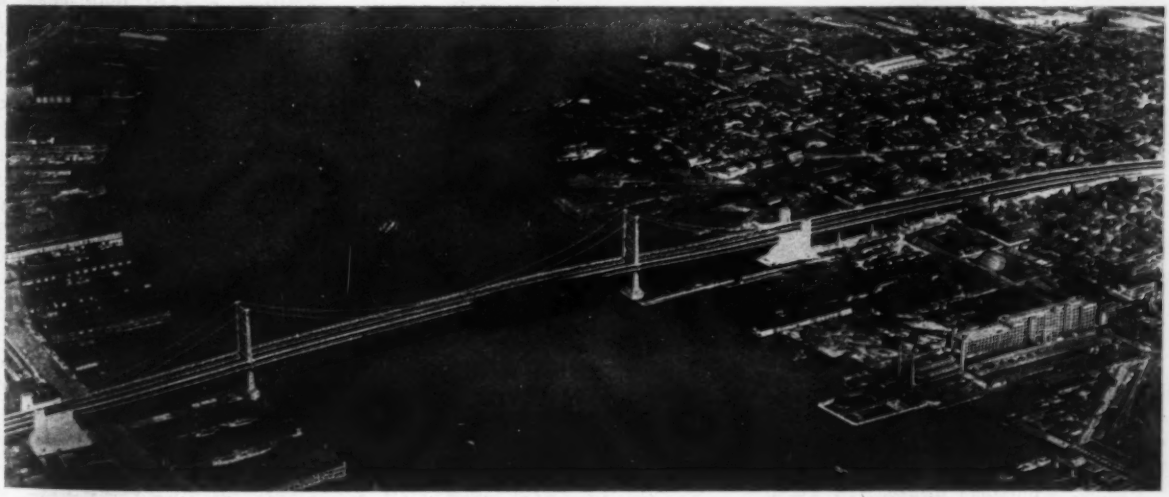


Fig. 2. Delaware River Bridge, Philadelphia, whose tower design was the first to account correctly for stresses from tower displacement resulting from cable pull.

Fig. 3. George Washington Bridge, New York, on which the great stiffness contributed by the cables themselves was first recognized. The deck, shown under construction in 1930, is stiffened only by the top chord of a future stiffening truss, required when a lower level is added.

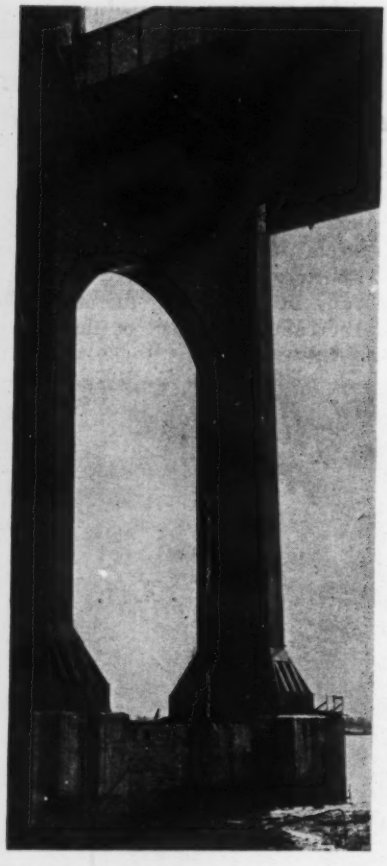


Fig. 4. Whitestone Bridge, New York, on which the need for only a light stiffening truss is recognized by use of a shallow plate girder.

Washington Bridge, 39,000 lb. per lin. ft. in its final condition, strikingly brought home to the engineers the seeming paradox that *the cable is the stiffest member in a suspension bridge.* The truss of this bridge will be, when fully completed, only 29 ft. deep or 1/120 of the span, and this because it is to be a double-deck bridge; the upper chord of the future truss forms the present stiffening girder and has been found perfectly sufficient for the upper-level highway traffic.

The conception of the cable as controlling and supplying the stiffness of the bridge is a very fertile principle, and it is in this direction that future advances in suspension bridges will be made.

The Golden Gate Bridge, with a span of 4,200 ft., is at present the longest-span bridge. Here the stiffening trusses were made still shallower, 25 ft., or 1 in 168 of the span.

Plate girder stiffening "trusses"

Making the stiffening trusses less stiff, and therefore of less depth, leads from articulated trusses to plate girders, which offer many structural advantages for connections and fabrication, present a simple and good appearance and are easy to maintain. The dimensions—2,300-ft. span and 74-ft. width—and the loads—3,000 lb. per lin. ft. live and 11,000 lb. dead—of the recently completed Whitestone Bridge in New York City pointed to a shallow stiffening girder that would also act as an economical wind chord. The bridge was stiffened by flattening the cables to a versine of 200 ft., thereby increasing the rigidity of its polygon, and making it possible to use a girder 11 ft. deep.

The Tacoma Narrows Bridge, now under construction over the Puget Sound, presented a similar but more extreme case. Because of foundation conditions a very long span was the only economical and feasible solution. At the same time, the anticipated travel could not sustain more than the cost of a two-lane bridge, and relatively small funds were available for it. To meet these difficult conditions a two-lane bridge was designed for a live load of 1,000 lb. with a span of 2,800 ft. and a width of 39 ft. The resulting weight of the bridge is 6,000 lb. per ft. It will be noted that the length-width ratio is 72 to 1. The required relative stiffness of the bridge is obtained in this case by a versine of the cables of 232 ft., or 1 in 12.1. These dimensions and loads made it possible to use as stiffening girder a plate girder only 8 ft. deep.

The problem of lateral forces

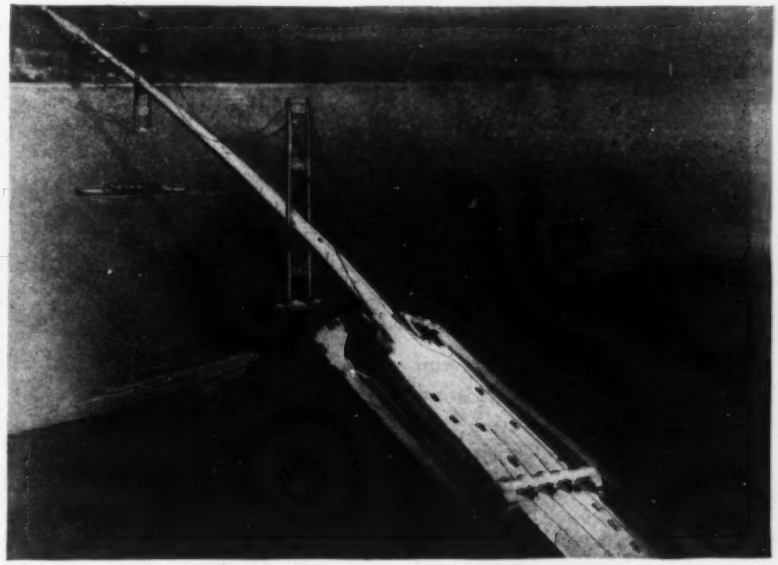
From the above recital of the de-

velopments in recent suspension bridges it will be noted that while the knowledge of the behavior of suspension bridges under live load and temperature variations increased greatly, knowledge of behavior under the action of lateral forces, such as wind, remained primitive. In the design of the earlier bridges, with a low ratio of length to width, a specified lateral wind load was applied on the trusses, and the stresses were determined as for a simple horizontal beam without any assistance from the cables. For the shorter spans and greater widths this crude method caused no inconveniences. For the Manhattan and Delaware bridges an approximate method was developed which took into account the relief offered by the cables to the trusses. With the advent of the much longer bridges of more recent years the problem of lateral stresses became acute.

Engineers had felt for some time that the standard rules for allowable length to width for truss spans do not apply to long-span suspension bridges. The problem was finally analyzed, and the so-called Elastic Distribution Method was developed. (See "Suspension Bridges Under the Action of Lateral Forces," by Leon S. Moisseiff and Frederick Lienhard, *Trans.* Am. Soc. C. E., 1933.) Fortunately an available model of the San Francisco-Oakland Bay Bridge at the University of California made it possible to verify the method experimentally. This method gives results which are more accurate than are our knowledge and assumptions of the magnitude and distribution of wind forces on bridges. It has enabled engineers to plan long spans more boldly, and at the same time to design them more economically so that they become practicable.

The design of recent suspension bridges shows that in the longer span highway structures the importance of the stiffening truss has been decreasing until the truss has become a shallow plate girder, designed for light live loads, whose function is to serve as a distributor of the loads over a small portion of the span only.

But while it was losing its structural importance as a beam for distributing live load and limiting deflections, its role grew more significant as a chord of the horizontal wind truss to resist lateral wind forces. This was due to the fact that the span of suspension bridges has been growing longer but their width has not increased in proportion. On the contrary, in several instances of lighter traffic bridges widths have been made narrower. The ratio of length to width thus becomes higher, resulting in higher wind stresses in the trusses which, in some instances, exceed the stresses due to live load.



Washington Toll Bridge Authority Photo

Fig. 5. Tacoma Narrows Bridge over Puget Sound whose large length-width ratio requires special strengthening of the web of its shallow stiffening girder to resist lateral wind forces.

Acting as chords of a wind truss, the stiffening girders are subjected to both axial tension and compression, while acting as stiffening girders they are subjected to bending stresses. The designer is, therefore, confronted with the problem of a girder simultaneously subjected to compressive and bending stresses.

With the introduction of plate girders, the problem turns to the stability of the web plate and its efficient and economical design. Either the girder web has to be made thick enough to avoid buckling, or it has to be properly stiffened. In the Whitestone Bridge the problem was solved by using an $\frac{11}{16}$-in. web plate for the 11-ft. deep girder, and stiffening it longitudinally by two angles between vertical stiffeners. Without these longitudinal stiffeners a plate thickness of $2\frac{5}{16}$ in. would have been required.

In the Tacoma Narrows Bridge a $\frac{1}{2}$-in. web plate will be used for the 8-ft. girder and will be stiffened by three longitudinal zees on one side of the web and vertical stiffener channels on the other side. Otherwise a plate thickness of $1\frac{3}{8}$ in. would be required.

More suspension bridges are being built today than ever before. In earlier days horse-drawn vehicles and street cars were not numerous enough to pay sufficient toll to maintain and amortize an expensive bridge. The advent of machine-driven vehicles and their rapid increase in numbers brought a change. Toll from vehicular traffic became feasible and profitable, making self-liquidating toll bridges practicable. Furthermore, automobile traffic can take relatively steep grades and its load concentrations are moderate, two conditions which make suspension bridges exceptionally suitable for modern service.

The development of suspension bridges in recent times has been in the direction of longer spans to meet the demands of traffic and to cross distances heretofore impossible to bridge. This demand is forcing engineers to meet the problems of efficiency and economy put forward by long spans. The direct way to successful solution of these problems lies in the application of scientific study to the available research of materials and to the behavior of models. It leads to a full utilization of our mathematical apparatus and to further research tests, and will result in bridges in which the advantages of structural form and of physical qualities of the materials will be more fully realized.

Gridiron Sodding for Road Fill

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Contents in Brief—Unusual use of sod lining for shoulder and down-slope gutters on road fills described. Strip sodding and seeded panels used to protect areas between gutters.

SOD GUTTERS and strip sodding on a sizeable fill slope in northern New York state have been successful in resisting erosion from several heavy rains and growth is developing nicely even with prevailing dry weather. The accompanying view and sketch indicate the general character of the protection work; the sodded gutters are uncommon.

The area to be sodded was a fill 700 ft. long and 25 ft. high at the center, with slopes up to 50 ft. long. There was to be a 4-ft.-wide sod gutter on the outer edge of the shoulder, parallel to the center line of the pavement, with 3-ft.-wide sod runoff or down-slope gutters, 20 ft. c. to c. at right angles to the pavement, and 2-ft.-wide strip sod, 10 ft. c. to c. between runoff gutters and parallel to the pavement. The areas left in between sod strips and gutters were of topsoil which was seeded. The arrangement is indicated by Fig. 2.

At the start, the slopes were trued to a 1 on 1¾ slope by dragging a 20-ft.-long 10-in. I-beam at an angle along the slope, using two trucks, one to pull through a Y-hitch and the other acting as a guy to hold the beam on the slope and control the angle so as to cut high spots and move dirt to low spots. High points too hard for the beam to cut were loosened with a pick and large stones removed by hand. After a dozen passes in each direction the slope worked into a plane ready for easy hand trimming. Excavation for the sod gutters that ran down slope was made in the subsoil using a template to get the proper shape. The gutters were 6 to 8 in. lower in the center than on the edges, Fig. 2.

Enough topsoil was dumped over the top of the bank to make 4 in. in depth over-all. The I-beam process was repeated, the hitch being adjusted so as to cut the soil down the bank. This also packed the soil. Hand trimming the finished slope was a comparatively easy job, redigging the down-slope gutters and excavation for the strip sod being the most work. Excavation for the strip sod was made to a depth of 2 in. leaving 2 in. of topsoil, the topsoil removed being

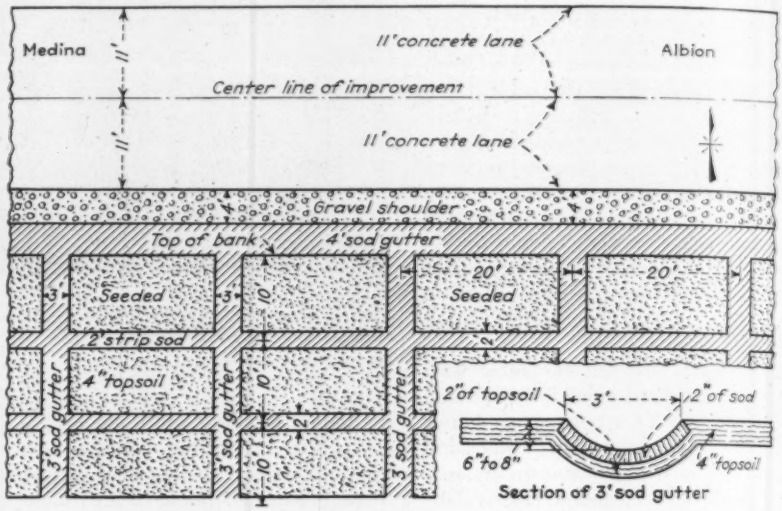

Fig. 2. Developed plan of fill slope shows interesting adoption of sod gutters, with strip sodding and seeded panels, for preventing erosion.

Fig. 1. A fill slope in New York state has a grid of sod strips and sod-lined gutters with seeded panels.

Fig. 3. Rolls of sod were slid down metal chutes to prevent tramping the smoothed topsoil cover.

spread evenly over the area to be seeded. The redigging of the downslope gutter also left 2 in. of topsoil under the sod.

As the excavation for strip sod and the sod gutters was completed, the entire area was fertilized at the rate of 2 lb. per 100 sq.ft. with 4-16-4 commercial fertilizer. As it was necessary to do this by hand on the slopes, small areas were laid out and computed so as to insure the right amount of fertilizer being spread. The area was then raked and left to stand 48 hr. before starting seeding and sodding.

Sodding started at the bottom of the runoff gutters, the strips being laid as the runoff gutters were brought up. The 4-ft. sod gutter on the edge of the shoulders was laid last so as to avoid trampling the newly laid sod. All sod on the slopes was staked on 1-ft. centers using 1x1-in. x 2-ft. stakes. Instead of carrying sod up or down the slope for laying, a semicircular metal chute (Fig. 3) was used, being laid adjacent to the gutter and the sod rolls unloaded from the truck and slid down the chute. The laborers were able to pick out the sod rolls right where needed, thus cutting foot traffic on the slope to a minimum. The chute was moved for each line of gutter.

The seeding of the slope was done after the sod was in place. A mixture of grass seeds was sown at the rate of 150 lb. per acre or 3.44 lb. per 1,000 sq.ft. This was done in small areas by hand and the seed then raked in and rolled. The rolling was done by using a winch truck equipped with a snatch block to pull a lawn roller up and down the bank. Two men were needed to guide the cable and move the roller for the lap. A small engine and pump mounted in a 1,500-gal. tank truck supplied water under pressure for sprinkling before and after sodding.

By these methods over 2,200 sq.yd. of sod was laid in eight working days with twelve men: Three men cutting and loading, using a power roller to pull the sod cutter; five men laying, staking, tamping, etc., and four men unloading and excavating for sod. William J. Gallagher of Medina, N. Y., is the contractor, and Frank Balcerzak is superintendent for the contractor.

Paint Maintenance on Bay Bridge

DURING the three years since completion of the San Francisco-Oakland Bay Bridge, which was opened to traffic in November, 1936, expenditures for maintenance, which was almost wholly painting of the steelwork, have been about $100,000 per year. No extensive or general painting has been required; it is expected that progressive spot painting and a program of general painting will bring the repainting cost ultimately to about $170,000 per year, which for 170,000 tons will represent a cost of roughly $1 per ton per year.

The general condition of the bridge to date is excellent. No structural maintenance has been required except for replacement or repair of some fender timbers around the piers, chiefly because of damage by vessels. The concrete floor slab of the bridge is in excellent condition; the quality of the concrete was high, and such cracks as have developed are practically invisible. No repairs have been required to the floor. It should be remarked that chains are not used on any of the bridge traffic. The total traffic, however, is high, the 1938 total being 8½ million toll vehicles or about two-thirds the traffic over the Philadelphia-Camden bridge in the same year.

The general condition of the paint remains good. Such breakdown as has occurred has resulted from salt spray on the east end of the bridge where the distance from the water to the steel is small, and on the floor system supporting the upper deck over the trucking lanes (north side of bridge) of the lower deck, which attack is attributed to the sulphur content of the exhaust gases from diesel-powered trucks. Some trouble is also attributed to airborne sand from the dunes at the west side of San Francisco which also carries some salt.

It is noticeable that the worst spots are on horizontal surfaces that are not washed by rain, especially the underside of curbs and walkways where considerable condensation occurs. Points of failure are thoroughly cleaned with wire brushes and spotted with two coats of red lead, after which the entire member is given a coat of aluminum paint. Maintenance bridges for each of the six West Bay suspension spans and the 1400-ft. East Bay cantilever were made a part of the superstructure contract. Three bridges which can be swung under any of the other spans are now under construction, as well as two gantries spanning the lower-deck truck roadway (each with a retractile arm which operates in the space over the railroad catenary) and scaffolds for tower painting.

Water Pressure Indicators For Use in Earth Dams

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Pressure cells with sensitive gold plated diaphragms replace wells in obtaining saturation lines and other hydraulic conditions within earth dams

IN ORDER TO BE INFORMED continuously and accurately concerning percolation within its large earth dams the U. S. Bureau of Reclamation now measures pore pressures in embankments with diaphragm-type hydrostatic pressure indicators. Each indicator measures the pressure of the percolating water at one point in the dam, and from these pressure measurements, the saturation (zero-pressure) line, the stream lines and the frictional forces of the flow are derived.

Previous to the use of these indicators, this information has been sought through observations of water surface elevations in wells. Such tests proved unsatisfactory due to the low percolation rates of earthfill materials. Even small wells of 1-in. diameter required large flows through the embankment for their operation, caused high percolation gradients near the well openings and modified the very pressures the wells were supposed to measure.

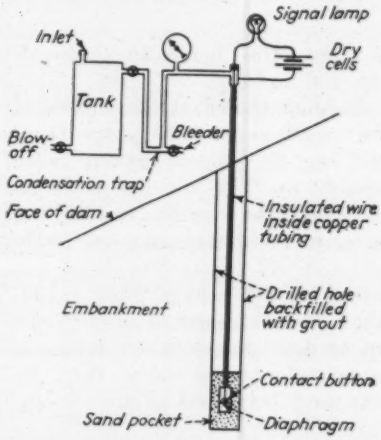

Fig. 3. Diagram of indicator as set up at a dam to determine water pressures.

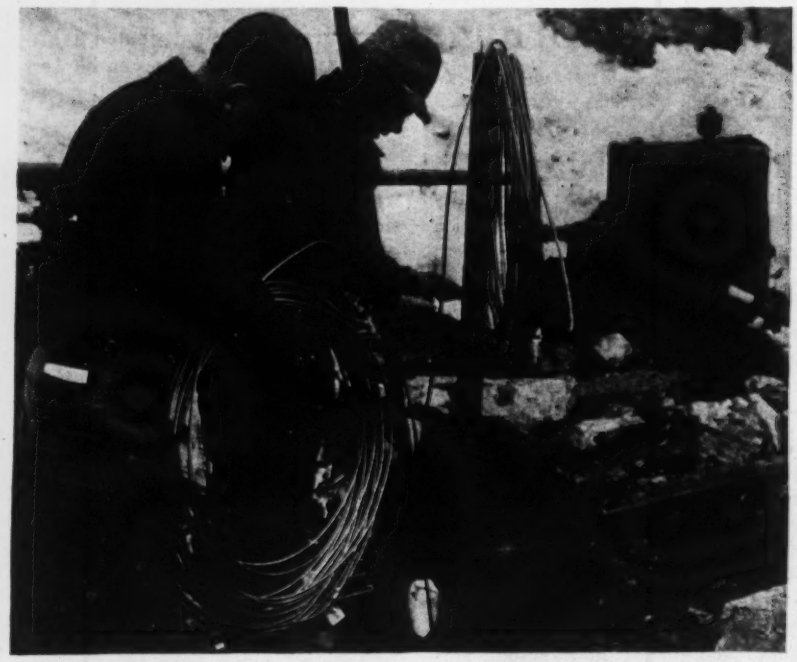

Fig. 1. Placing an indicator (in bag of sand) in perforated pipe sunk in a drilled hole made by a well drilling outfit after the dam has been completed.

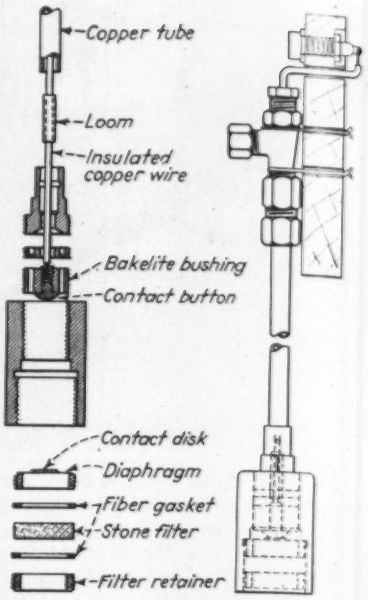

Fig. 2. Parts of the indicator and the method of assembly.

To minimize influencing the natural water pressures in the embankment a shift was made to hydrostatic pressure indicators using a modification of the diaphragm principle of the Goldbeck earth pressure cell. With the soil pressure held back by a porous disk, water pressure in the embankment holds a thin metal diaphragm up against a contact button within the indicator. During a test the air pressure in the indicator, which is measured by a gage at the test set, is gradually increased until the diaphragm is forced away from the contact. This broken contact extinguishes a lamp at the test set and the gage pressure at that instant equals the water pressure at the indicator.

The indicators are 1 in. in diameter and 2 in. long, have gold-plated diaphragms of 0.0015-in. brass and are connected to the test set by $\frac{5}{16}$-in. copper air tubes inclosing insulated copper wires. The indicators are sensitive to pressure differences of less than 1 in. of water, although such sensitivity is not required when they are installed.

In a typical installation about thirty indicators are distributed

through each of several cross-sections of the dam, with separate tubes leading to terminal boards at the crest. To avoid interference with the earth-placing operations, the installations are made in holes drilled in the completed embankment, several indicators being placed one above another in each hole. Each indicator is surrounded by a small sand pocket and the rest of the hole is backfilled with grout, thereby making each hole the equivalent of several individual wells opening at different depths.

Pressure measurements are being taken at least once every two weeks. These pressures are plotted on the cross-section of the dam to locate the equal-pressure lines, or are combined with their indicator elevations to locate the corresponding equipotential lines and the stream lines normal to them. Due to the fact that the installations are so recent, data on complete cycles of saturation and drainage of the embankments have not yet been accumulated, but the present successful operation of the indicators promises significant information as to conditions within the dams.

Condensation and Building Insulation

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***Contents in Brief**—Danger of moisture condensation in insulated walls can be eliminated by vapor-proofing the interior surface; merely vapor-proofing exterior surface, as with building paper, accentuates difficulties, according to National Mineral Wool Association tests. From a paper given at a meeting at the New York World's Fair.*

ONE OF THE LATEST properties of mineral wool to receive scientific investigation is that of moisture resistance. While a single test will demonstrate that mineral wool is not hygroscopic and therefore will not absorb moisture from the air, the moisture problem as developed in recent years is too complicated to be settled by this simple test. Other factors enter into the problem, such as high relative humidities required in modern air conditioning, new types of building construction, arrangement of materials in a structure, insulation used and temperatures encountered.

The National Mineral Wool Association, recognizing the need for knowledge on basic principles, conceived of a research program to investigate the causes of and methods for preventing condensation. This program has been supported entirely by the association and carried out under the direction of the University of Minnesota for the general benefit of the building public and of those involved in air conditioning, building construction, and the manufacture of building materials.

Why condensation occurs

As is well known, all air contains moisture, and the amount of moisture which may be contained in a given volume of air increases as the temperature increases. If air containing moisture is cooled it will ultimately reach a temperature at which it is saturated and below which moisture will be precipitated. This is said to be the dew point temperature of the air, and is illustrated by the condensation of moisture on the surface of a glass of cold water. The moisture difficulty in buildings has been brought about by allowing moisture in the form of water vapor in the air to come in contact with materials whose temperatures are below the dew point temperature of the vapor. When the relative humidity of air within a building is raised, the dew point temperature is raised and there is more chance that condensation will take place.

In recent years relative humidities have been increased by two methods, first, by artificial humidification as added in many air conditioning systems, and second, as a by-product of better building construction. The addition of weatherstrips and better wall construction prevents the exchange of air through the exterior walls of a building, which is the usual method of dissipating the humidity created within a building by normal living processes. Water vapor does not confine itself to contacting the interior surfaces of the walls. It penetrates through many types of building construction and, in passing outward through an exterior wall, will come in contact with colder materials which may cause condensation within the wall even though there was none on the interior surface.

Under normal conditions the moisture content will be graded from the inside or warm side to the cold side of a wall, the density at various points depending upon the resistance of various parts of the wall to the passage of moisture. High vapor resistance on the inside surface and low resistance on the outside surface of the structure means low vapor density within the wall, whereas, with the opposite conditions, high vapor densities may be created with greater possibility of condensation.

Wall construction at fault

It has been common practice in frame wall construction to add a building paper on the outside sheathing to prevent the penetration of wind and free moisture. In recent years building papers have been developed which are not only moisture-proof but also vapor-proof. When such materials are placed on the outside of a wall with no particular vapor-resisting material on the inner surface the vapor density within the wall will be increased and a dangerous condition result.

When insulation is added to a wall it stops the outward flow of heat

and decreases the temperature of the outside portion of the wall. If the vapor conditions within the wall are not changed the lower temperatures will increase the possibilities of condensation. For this reason insulation has sometimes been considered as the cause of condensation in cold climates.

Facilities for testing

In studying the moisture problem at the University of Minnesota, it was desired to get as close as possible to practical operating conditions, that is, full-scale building construction, with normal inside air temperatures and humidities and extreme low outside air temperatures. For this purpose a test room was built 30 ft. square and 25 ft. high, insulated with 6 in. of mineral wool and provided with cooling equipment to hold the air temperature down to —25 deg. F. Small-scale test houses were constructed to meet given humidities and temperatures. Many different combinations of materials were used within the walls of the test houses, and they were constructed so that they could be taken apart and inspected during test periods without interrupting the tests. Conditions were varied but in many instances the test periods were continued for twenty days with outside air temperatures of —20 deg. F.

Use inside wall protection

The results of the investigation may be summed up briefly as follows:

In most types of building construction vapor will pass through the wall in direct proportion to the vapor pressure differences across the wall, and in inverse proportion to the vapor resistance along the path of vapor flow. For new construction the best solution of the problem is to prevent the vapor from entering the wall by using some vapor-resisting material on the inner wall surface. There are many good vapor barriers in the form of asphalt or metal coated papers, although all asphalt papers are not good barriers. The materials used on the outside surface of the wall should preferably not have a high resistance to the passage of vapor.

Trouble is rarely met in old construction unless excessive humidification is carried. Where it arises it is possible to add a vapor barrier, in the form of oil or aluminum paint, to the inside surface of the wall. Usually the results obtained by venting to the exterior are secondary to those which may be obtained by using a vapor barrier on the interior of the wall.

The most effective method of preventing condensation within a wall is to reduce the relative humidity of the air within the building.

The causes of condensation within the walls of a building are comparatively simple and the remedies may be applied without sacrificing any of the elements of good building construction, or of desirable air conditioning within a building.

Supplementing the moisture problem studies the National Mineral Wool Association has attacked in a practical way the relation of insulation to comfortable summer living conditions. Two test houses have been constructed in Minneapolis and two full-scale houses have been built in New York. Of each pair of houses, one will be left without insulation and one will be insulated with various thicknesses of materials throughout the upper floor ceiling and side walls, the insulation being changed to get as many different combinations as practicable for comparison with the uninsulated house. Approximately 50 thermocouples will be used in each house. Continuous records will be kept of sunshine, air temperature, wind velocity, and wind direction. These tests are now under way and give promise of the same practical results as obtained in the condensation study.

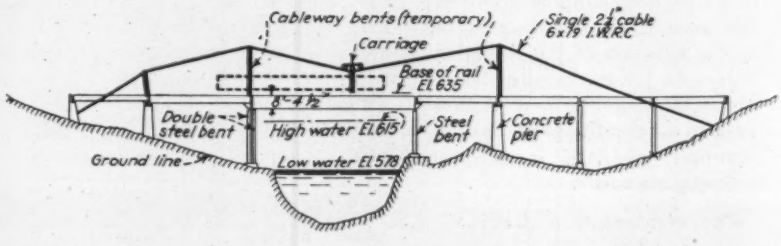
Cableway Places Steel Girders

THE CENTRAL SPAN of a combined railroad and highway bridge, built for construction purposes across the Sacramento River just below Shasta Dam, has a length of 115 ft. The river channel was subject to floods at the time of erection and hence falsework would have involved some risk. This was avoided by the use of a cableway arranged as shown in the accompanying sketch. The steel girders for the shorter spans were put in position by one of the contractor's "cherry pickers." When the structure was ready for the long channel span, bents of structural steel were erected on the piers at either end of the span and the 2¼-in. cable was strung.

The long girders were unloaded from flat cars to the bridge deck as near final position as possible. Thence, with the aid of the cableway, each girder was slid to within about 2 ft. of final position. The end of the girder was lowered to place with the aid of a sling over the cableway carrying a three-part line.

Steel bents were used for the channel piers because they were less expensive than concrete. However, in addition to the abutments, it was thought advisable to use concrete piers, one span back from the river channel on either side, to provide anchorage.

The bridge is part of the work under contract to Pacific Constructors, Inc., for whom F. T. Crowe is general superintendent. Ralph Lowry is construction engineer for the U. S. Bureau of Reclamation.



Steel girders for the central span of this Shasta Dam construction bridge placed with the aid of the overhead cableway.

Before You Build an Incinerator

HENRY W. TAYLOR
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***Contents in Brief**—Conditions which are entirely local in character have an important influence on refuse incinerator design. Preliminary investigations, therefore, should include a study of collection methods along with careful determinations of the quality and quantity of material to be burned.*

INCREASING RECOGNITION is being given to the necessity of making a preliminary examination of local conditions affecting an incinerator project prior to the determination of size, type, arrangement and facilities to be included. These local conditions introduce important requirements, and the design of the incinerator must be adapted to these predetermined factors and their peculiarities. In a sense, it may be true that rubbish is rubbish and that tin cans and ashes are more or less the same the country over. However, mixed refuse differs widely in the percentages of various constituents, and the resulting mixtures are not the same nor can assumptions be made on the basis of wide generalizations.

Preliminary investigations should include a review of the residential, commercial and industrial characteristics of the community. Accurate knowledge of the collection system and the materials collected will often lead to adaptations in method to suit better the requirements of sanitation and incineration. For instance, where ashes from schools, churches and other institutions are now collected with other material and would normally be delivered to the incinerator, a slight variation in collection will make it possible to deliver this material directly to the dump. A modification of this kind will reduce the amount of inert material which would otherwise be taken to the incinerator and again handled as residue from the incineration process.

Actual tests should be made in order to obtain accurate data as to the quantity of refuse, and the minimum, average and maximum loads, together with the time and the frequency of such variations. And where collections of mixed refuse are made from a part of the total area of the district on alternate days, it is necessary to take into account the daily change in the character of the material which will be collected from residential and commercial areas. Information of this kind has a pertinent bearing on the capacity and design of the incinerator and its equipment.

Components of refuse

The smaller the installation, the more important is the determination of the type of mixed material to be burned. Peak loads or a variation in quality which are readily absorbed in a large plant will have a pronounced effect upon a small installation. Therefore, the characteristics of a mixed refuse must be determined. The first step consists of subjecting test loads of the refuse to sorting into various recognized classifications. After the percentages of these physical classifications have been determined, it is then possible to approximate the available combustibles and the general chemical characteristics of the mixed refuse.

For practical purposes four classifications suggest themselves, and these are given below with comments:

Metals, Glass, Cans, etc.—These are non-combustible and in large measure non-putrescible. The paper and food in cans is ignored as having no heat value.

Coal Ashes—These are non-putrescible and their heat value, except for power production, may be ignored. The tendency of this material to form slag at higher temperatures cannot be ignored. It may, however, be assumed that there is a sufficient heat value in coal ashes to supply the required heat to raise its temperature without robbing heat from the more combustible materials.

Domestic and Industrial Garbage—Delivery of industrial garbage must

Fig. 1. Refuse differs so widely in composition that field sampling and classification analyses are an essential prerequisite to incinerator design.

be subject to control and regulation in all cases where industrial establishments may be possible contributors of such material.

Combustible Rubbish—Data as to the contributing sections and periods of collection of the more combustible rubbish must be available for proper storage and suitable mixing of this material at the incinerator.

Sampling and testing

This process of sorting of representative loads of mixed refuse also provides an opportunity for practical sampling. Since the direct sampling of mixed refuse is practically impossible (and would probably be misleading) representative sampling of the classifications of similar material is recommended. Furthermore, while the weighing and sorting determinations are being made it is possible to carry on field analyses of sorted garbage and sorted combustible rubbish which will throw further light upon the problem.

Field determinations of chemical characteristics can be made with inexpensive and easily operated field equipment. For moisture determination, an oven of the type sold for use on top of gas stoves can be used. It should be insulated, and fitted with a ventilator at the top, an electric resistance on the lower tray and a thermostat. Samples of garbage weighing from 600 to 800 grams and those of combustible rubbish weighing about 300 grams are placed in the oven, which is kept at a temperature of 200 to 210 deg. F. The drying of this material is continued until a fairly constant weight is obtained; this operation sometimes requires from 16 to 24 hours.

For the distillation of volatiles and the burning of solids, a small portable gasoline furnace, with a fuel capacity of one gallon and equipped with a pressed steel pot can be used. To obtain the percentage of volatiles, the dehydrated sample is packed in the steel pot, covered with three rings of asbestos, and weighted down to exclude as much air as possible. The burner is then adjusted to maintain a discharge of volatile gases from the sample until this action ceases. After the sample has cooled to a temperature below that of ignition, the asbestos covers are removed and the sample weighed. It is then returned to the steel pot, which is heated redhot while the sample is well stirred until combustion is completed. The residue is weighed as ash.

Applying the results

Assuming that these field determinations represent all the chemical data which will be obtained, it is necessary to break down the total percentage of volatiles into their combustible and non-combustible component parts. The total volatile percentage includes volatile carbon, available hydrogen, nitrogen and additional moisture, which is considered as combined moisture and an integral part of the material. This moisture is not driven off at low temperatures in the drying oven and it is assumed that all the oxygen in the ultimate analysis was originally combined with hydrogen and existed in the form of either free or combined moisture.

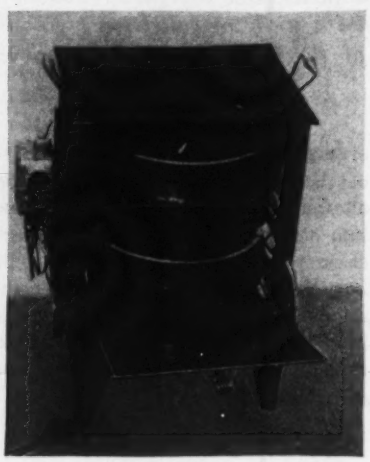


Fig. 2. Electric oven used for moisture determinations.

We are especially interested in the percentage of the total volatiles representing this combined moisture, and without the ultimate analysis, an approximation must be made based on the study of ultimate analyses of fuels such as coal, peats, and other materials for which ultimate analyses are available. Such a study of a large number of fuels of various characteristics has indicated that the volatile carbon and available hydrogen represent from 37 per cent to 72 per cent of the total volatiles. The analyses of materials approximating the characteristics of garbage and rubbish suggest an assumption of approximately 40 per cent. The combined moisture then plus the nitrogen gives a total of about 60 per cent of the total volatiles.

To illustrate the utilization of the field sorting data and field analyses, the following example may be taken from actual field determinations made prior to an incinerator design:

	Per cent	Per cent
Metals, glass, cans, etc.	8	
Ashes in refuse	15	
	23	
Sorted non-combustibles		23
Garbage (sorted)		28
Combustible rubbish (sorted)		49
		100

	Garbage Per cent	Combustible Rubbish Per cent
Moisture	71	24
Volatiles	17	54
Fixed Carbon	7	16
Ash	5	6
	100	100

Applying analyses to sorted percentages, we have:

	Per cent	Per cent
Sorted non-combustibles	23	
Ash (from analysis)	4	
	27	
Solid non-combustibles		27
Moisture (free)		32
Volatiles (total)		31
Fixed carbon		10
		100

Assuming 60 per cent of the volatiles is combined moisture and nitrogen and regrouping:

	Per cent
Solid non-combustibles	27
Volatile non-combustibles	51
Total carbon	20
Available hydrogen (estimated)	2
	100

It is apparent that the residue from incineration will be 27 per cent by weight; that heat must be available to raise the temperatures of this material together with 51 per cent of volatile non-combustible and that we have 22 per cent by weight in carbon and hydrogen to produce and maintain adequate temperatures to completely burn the refuse and raise the temperatures of the non-combustibles.

This field information obtained from sorting of mixed refuse and proximate analyses of parts thereof provides adequate data to form the basis of the design of the type and size of cell, the size of unit, the number of units and the capacities and adaptations of the various component parts of the incinerator to the determined character of the material to be burned. The local conditions govern the amount and type of handling apparatus and plant equipment best suited to the project. This latter consideration is one receiving much attention in recent designs and is a study of mechanical handling of material in and out of the incinerator. It represents a problem by itself outside the realm of combustion.

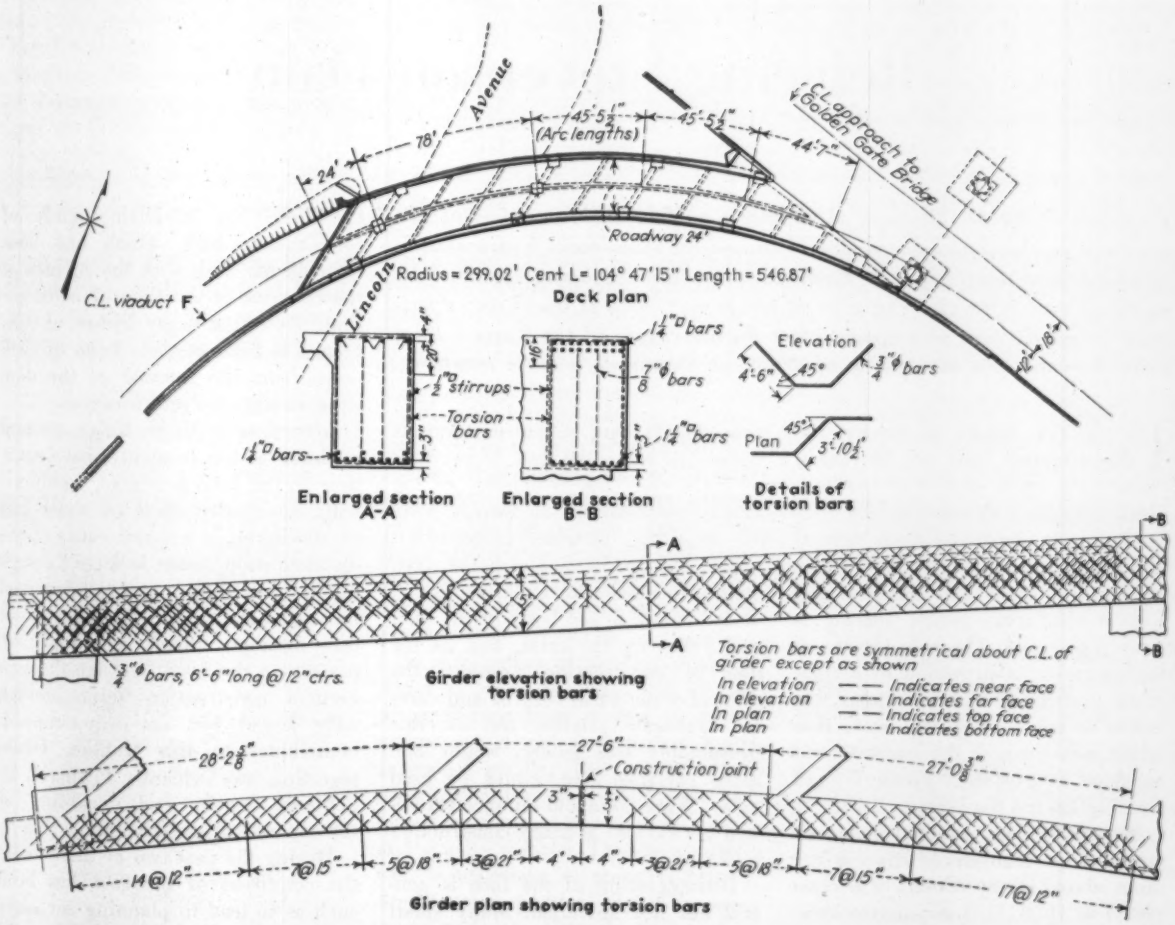
Curved Concrete Girders For Golden Gate Bridge Approach

A HIGHWAY VIADUCT whose concrete girders are curved in plan on a radius of 299 ft. is being constructed by the California Division of Highways on viaduct "F" of San Francisco's Funston Ave. approach to the Golden Gate Bridge. The relatively higher cost for curved girders, the highway commission believes, was justified because: (1) they maintain a constant spacing between girders from span to span; (2) the overhang from girder to outer edge of deck slab does not vary with the curved girders, while with straight girders this variation would be almost 3 ft.; and (3) as the underside of the bridge is visible, the curved girders add much to architectural appearance and harmony.

Three curved girders are used, one on the center line and one at each outer edges of the 24-ft. roadway. Curvature and spans are so proportioned that the straight line connecting girder centers at points of support always stays within the girders. In designing for the torsional moment due to curvature, it was assumed that 20 per cent of the torsion will be taken by the floor slab, which will be cast integral with the three girders. The remaining 80 per cent of the torsion was considered as resisted by spiral reinforcing in the girders. This reinforcing for all three girders totals only 7,000 lb. In working out the number and spacing of the torsion bars, the formula used was that developed by Prof. E. Morsch, in "Der Eisenbetonbau," published in 1922.

As to the additional cost for curved girders of this type, compared to straight girders; bid prices per cubic yard were $22 on viaduct "F," as compared to $17.60 bid at the same time on the three other viaducts of the Funston Ave. project. This $4.40 difference was not ascribed to girder curvature alone; other factors that increased the cost on viaduct "F" were a sharp superelevation, skew construction, and the fact that at one end the deck will be 80 ft. above the ground.

Plans for viaduct "F" were developed under the direction of F. W. Panhorst, bridge engineer, California Division of Highways and construction is being carried out by the department.



Plan of curved viaduct "F" in Funston Ave. approach to the Golden Gate Bridge and girder details showing location of diagonal reinforcing designed to carry torsion.

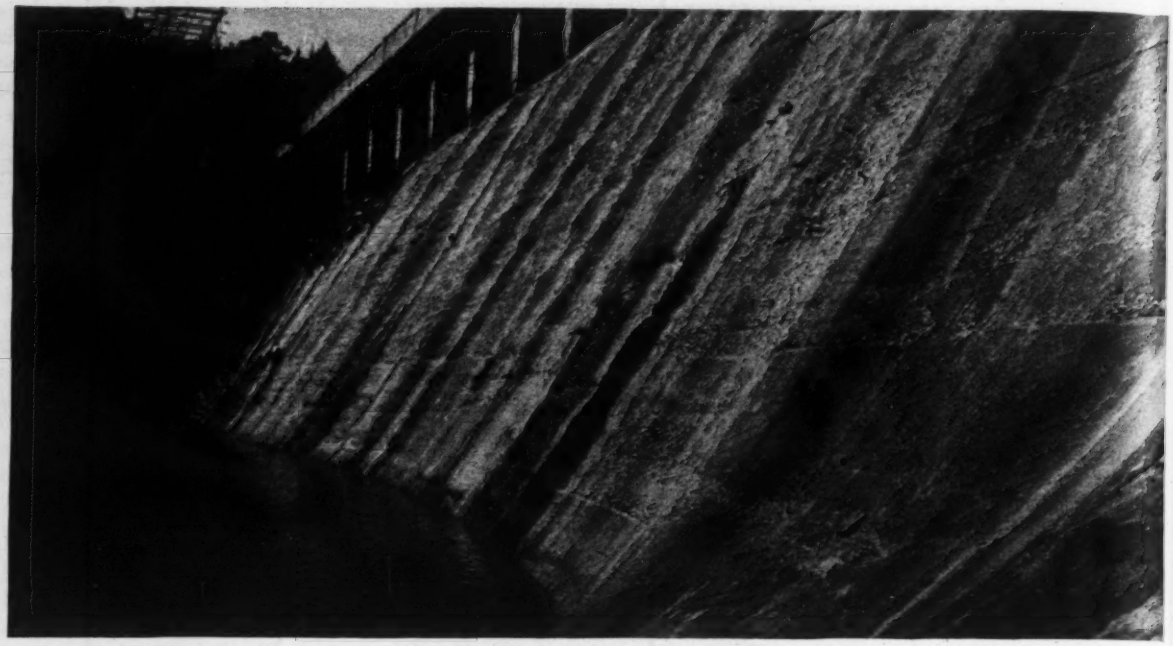


FIG. I. STEVENSON DAM ON THE HOUSATONIC RIVER IS BEING REFACED OVER A SYSTEM OF DRAINAGE CHANNELS.

Refacing Stevenson Dam

Contents in Brief—Because of extensive surface disintegration of the concrete on the downstream face of Stevenson Dam, main hydro station of the Connecticut Light & Power Co., the dam is being repaired by drainage and refacing. The plan of the repair work is based on experience in taking care of seepage water during repair of the Tunnel Dam near Norwich, but the system of drains at Stevenson is more complete.

IN TWENTY YEARS of service the downstream face of Stevenson Dam, main hydro station of the Connecticut Light & Power Co., has been eroded to a depth of from several inches to a foot or two by the action of weather, water and ice. Refacing with a reinforced gunite coating is now in progress. The new feature of the operation is provision of a complete system of drains under the facing to carry away any water that might leak through the concrete and develop hydrostatic pressure and freezing behind the facing.

Stevenson Dam, situated in a rock gorge of the Housatonic River a few miles above Derby, Conn., was completed in 1919. It is a concrete overfall dam of about 120-ft. structure height and 73-ft. hydraulic height, about 800 ft. long. A four-unit powerhouse lies below the dam at its south or right end; the remainder of the structure consists of 21 overfall bays 30 ft. long between construction joints and roadway piers. The concrete was made with bank-run gravel in 1:3:5 mix in the body of the dam and 1:2:4 on the crest, and as the time of construction preceded the days of water-ratio control and careful aggregate grading the concrete presumably was leaner, wetter and more variable than would be used today. Water gain and laitance formation are evident at many construction joints.

Disintegration of the face is general but not universal. Many small sections of the face still show the original form-board marks, while elsewhere the projecting ends of form-anchor bars, which had been chipped off flush with the surface of the concrete at the time of construction, show that many inches of concrete has been eroded. Tests of drill cores from the interior of the dam show strong concrete, however.

According to H. M. Kopp, general engineer of the company, ice formation on the face in winter, combined with the erosive effect of water and ice discharge, is a major cause of the disintegration. Some leakage through the flashboards occurs whenever pool level is above the crest, and in winter this leakage forms adherent ice masses on the face. Leakage through vertical construction joints, which were keyed but not waterstopped, contributes to this trouble. Disintegration was evidently furthered by deficient density and durability of the concrete at many points.

During the past two or three years the condition of the face has been such as to lead to planning for early repair. The rate of loss of material was increasing and, while the stabil-

ity and strength of the dam remained ample, it was thought inadvisable to let the process continue. Comparative estimates indicated that granite facing would cost about twice as much as refacing by gunite. The latter was adopted in view of its satisfactory performance on prior repair of dams and walls of the company's system, including Tunnel Dam near Norwich.

This latter job was considerably smaller but had to deal with more general corrosion. The dam is 31 ft. in maximum height and has about 28 ft. of head when the pool is at crest level. It was built in 1906, and in 32 years had been eroded badly on its downstream face. In the refacing job a horizontal drain trough was installed in a channel chipped in the face a foot or two below normal tailwater to collect water percolating through the structure and carry it off through a series of downspouts (see sketch Fig. 2). Expansion joints and vertical drains were installed wherever their need was indicated by cracks or leaks in the concrete surface, and weep pipes were placed at moist spots.

Conditions at Stevenson Dam were believed to call for more complete face drainage, in view of the greater height of the dam, the existence of vertical construction joints at frequent intervals, and a certain amount of distributed leakage, indicated by moist spots on the face. It was therefore decided to drain on the line of each vertical construction joint and to intercept leakage through horizontal construction joints and through the body of the concrete itself by a herringbone system of inclined drains discharging into the vertical drains (Fig. 4).

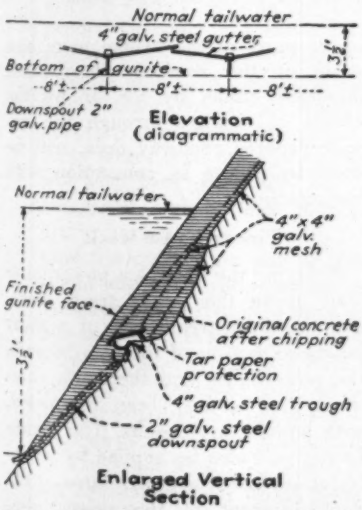


Fig. 2. Drainage near tailwater at Tunnel Dam was provided by a single horizontal gutter with drain pipes.

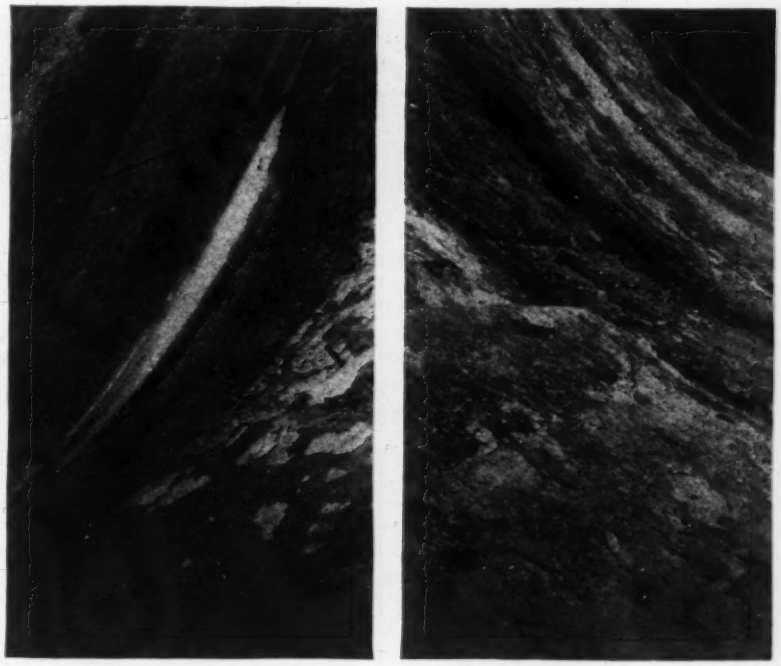


Fig. 3. At some points the original surface of the concrete is still visible while other places are eroded 3 to 6 in. deep as shown (left) by form-anchor bars cut off at the face during construction. Right view shows erosion at construction joints.

The refacing procedure involves the following steps: (1) Chipping the face to hard concrete; (2) cutting the V-shaped drain channels, applying a protective cover strip of elastite and (in the case of the vertical construction joints) placing a copper expansion-joint strip over the elastite; (3) placing 4 x 4-in. heavy-gage galvanized steel mesh over the face and anchoring it to dowels set in the concrete; (4) shooting on the gunite. The reinforcing mesh is carried continuously over the inclined collecting troughs, but is clipped over the copper expansion-joint strip at the vertical drain channels.

In application of the facing no attempt is made to build out to the original face line, as this in many places would call for two or more layers of gunite. The general guniting is therefore done only to the extent of leveling up to reasonably even and smooth surface. Thicker application is necessary at some points of deep erosion, particularly at the construction joints. Here the first coat is kept down to what experience has shown to be a practical maximum, namely 3 to 4 in., and over this a second layer of reinforcing mesh is applied and anchored and an additional coat shot, followed by others where necessary to bring the coating up to a proper surface.

Contract for the facing work was let to the Cement Gun Co., Allentown, Pa., on the basis of a fixed sum for plant installation and a unit price per

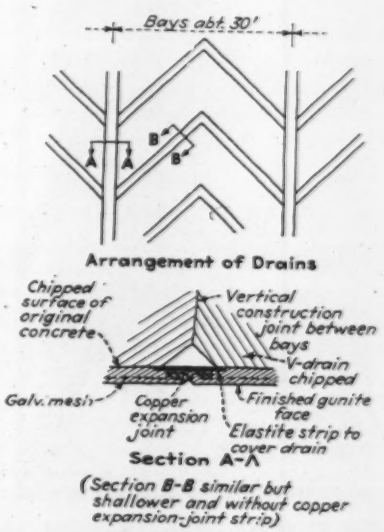


Fig. 4. Arrangement and construction of face drains, Stevenson Dam.

bag of cement to cover all costs of chipping, application of reinforcement and shooting. Frank Kurzeja is foreman on the job for the contractor. The work is done from traveling scaffolds suspended from anchors set in the crest of the dam, and proceeds panel by panel from the south or powerhouse end of the dam to the opposite end. Chipping and cutting the drain channels proceeds in two or three bays at a time, while in the next bay back the wire mesh is placed and in the preceding bay or bays gunite is shot. Flat-blade pneumatic chisels are used in chipping. Completing one bay takes 7 to 8 working days.

The mixer that supplies the gun is set up on the end of the dam at the level of the powerhouse floor, and the dry sand-cement mix is hauled from here to the gun set on the roadway that crosses the dam, near the panel that is being refaced. The mix is dumped into a hopper over the gun, from which in turn the gun is charged. An excellent sand is obtained from a pit near the dam.

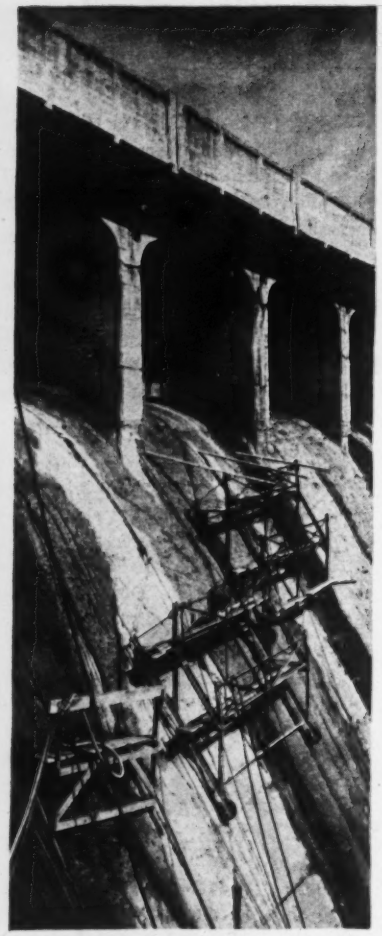


Fig. 5. Rolling platforms hung from anchor bars at crest give access to full area of face.

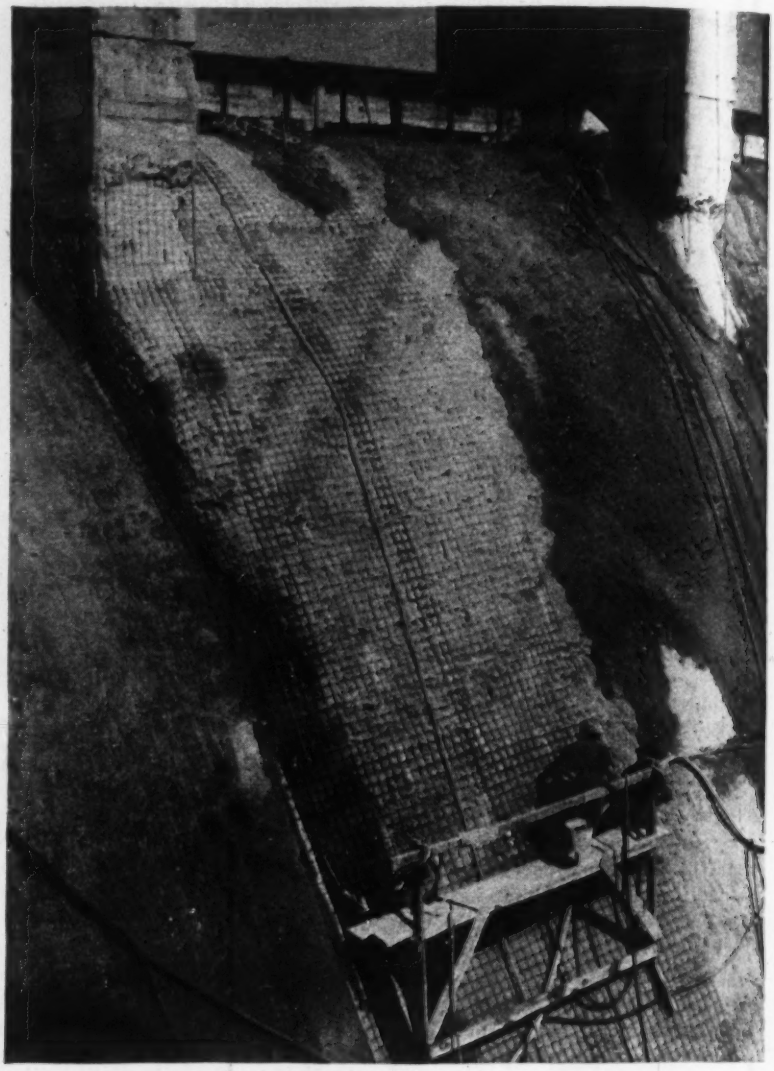


Fig. 6. Refacing is done after chipping the surface to hard concrete and cutting vertical and diagonal drains. The gunite is shot on galvanized wire mesh.

No edge forms are used, shooting being guided by eye alone. The surface of the gunite is brushed with a whitewash brush soon after shooting, which produces a texture rather smoother than the belted surface of a concrete road slab. The applied gunite is cured for two or three days by use of a water spray or wet burlap. The facing will ultimately be carried down below minimum tailwater level, which is about the top of the rollway.

It is expected that about a third of the work will be carried out during the present season, the remainder to be done over several years in the light of experience with the finished part of the work. The face work involves about 62,000 sq. ft. of gunite facing. Some accessory repair work on the lower parts of the bridge piers and some of the roadway floorbeams, damaged either by ice or by the effects of leakage through the unwaterproofed roadway deck, will be done by gunite in connection with the refacing.

Power house work

Refacing the upstream powerhouse wall above the intake trash racks, and the downstream face of a short bulkhead section of the dam between the powerhouse and the south abutment, has already been completed, with satisfactory results. The gunite facing will also be applied to the interior of five sluiceways through the dam adjacent to the powerhouse, whose $\frac{3}{8}$-in. steel plate lining has collapsed or been partly destroyed in the twenty years of service of the dam.

Grouting With Chemicals

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Contents in Brief—Abroad, consolidation of soils by chemicals, usually sodium silicate in combination with calcium chloride or some heavy metal salt, is practiced extensively. Early difficulties in controlling time of set have been overcome. The two chemicals are now injected simultaneously instead of separately. Chemical grouting is particularly useful in soils too fine for other types of grout to penetrate.

THE CONSOLIDATION of ground by grouting has undergone many changes in practice. The old terms of grout and grouting have ceased to mean specifically a liquid mixture of cement and water and the process of injecting it into soils and rock to make them watertight and strong. All sorts of water mixtures of clay, bentonite, asphalt and chemicals make up the grouts now used in ground solidification for engineering works. In this article grouts of chemicals are considered. These have increased in use remarkably in Europe but not greatly in the United States.

Permeability is greater

Chemical grouting makes its chief claim on its greater ability of penetration; it will penetrate finer-grained soils and smaller seams in rock. For example, no matter how wet cement grout is made it consists of solids suspended in water and the solids can penetrate no opening smaller than their particle sizes. Indeed, according to the theory of filters, a granular material consisting of grains ten times larger than those in suspension will keep back practically all of the suspended grains. Since the cement grains have a diameter of 0.004 in., sand having a grain size of 0.04 in. would effectively stop cement grout. Therefore, if a grout pipe were driven into sand finer than 0.04 in., the sand would act as a screen and hold back the cement. Increased pressure might make it possible to move some of the sand away from the pipe so that a bulb of grout would be formed there, but the cement itself would never get far from the pipe. In the case where a lens of coarse material lies adjacent to the pipe, the cement grout follows the line of least resistance and fills the voids in the lens while the rest of the area is not grouted at all. In the case of cracks and crevices in rock that are no wider than cement grains, cement grout cannot penetrate.

Chemical grouting is applicable to soils with a permeability of not more than 0.1 and not less than 0.0001 cm. per sec. for a liquid substance of the same viscosity as water at 60 deg. F. Such strata as clay or dust-fine sand or mud are essentially impermeable to water, their permeability being from 1×10^{-6} to 1×10^{-8} cm. per sec., and cannot be grouted.

How the chemicals act

Briefly, chemical grouting consists of filling the voids with two or more chemical solutions which harden upon contact with each other. One of the chemicals is usually sodium silicate and the other is a salt, such as calcium chloride or aluminum sulphate or any heavy metal salt.

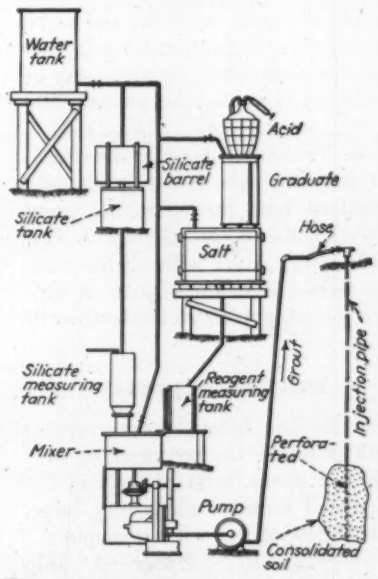

Fig. 1. Schematic arrangement of chemical grouting plant as it was set up for the operations at Agen, France.

Sodium silicate is a colloid, a substance which under certain conditions sets to form a gel. This gel is actually composed of microscopic solid particles in a liquid mass. It is sticky and semi-solid and gradually hardens as it loses some of its liquid content. Upon the formation of gel, the soil becomes impermeable. Upon the hardening of gel, the soil increases in strength and becomes stabilized, actually forming into rock. It can be subjected to heavier loads and can be excavated on a vertical face with picks or even by blasting.

The silicate of soda used for grouting should have a density of not less than 42.25 deg. Bé and a ratio of one part Na_2O to not less than 3.22 parts of SiO_2.

Development of processes

Silicate of soda was first used for chemical grouting in 1911. Two Belgians, Lamaire and Dumont, had trouble waterproofing and bracing a salt mine shaft. Cement grout was useless due to the aggressive water. Therefore silicate was applied, being immune to salts and acids, and it provided a waterproof sediment. The salty soil acted as electrolyte, the silicate gelatinized and therefore did not penetrate very far. To prevent sudden congealing sulphuric acid was added. However, the problem was too specific and was limited to the salty soil.

The next step took place in 1914 when two methods, the Joosten and the Francois, were developed using sodium silicate and a reagent, one calcium chloride, the other aluminum sulphate. Both methods consist of injecting the silicate as the grout pipe is driven into the soil; this injection presses out all the water and fills the voids, then the salt solution is injected during the raising of the pipes.

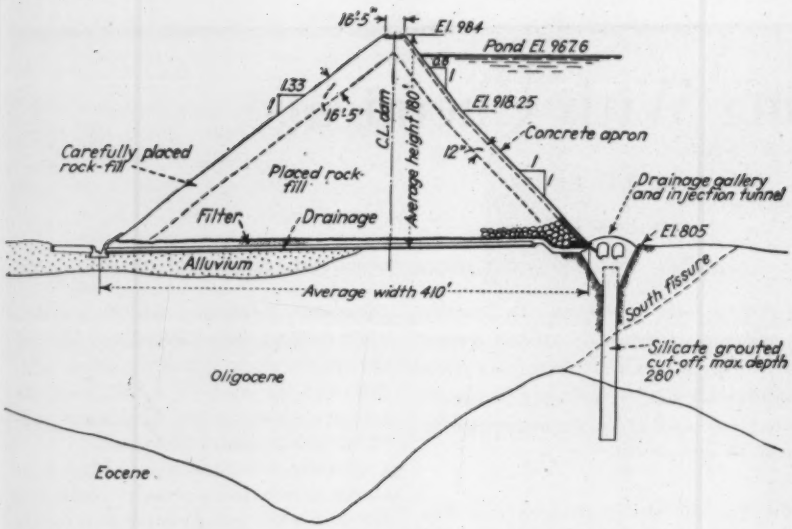

Fig. 2. At Bou-Hanifia Dam in Algeria a deep cutoff in line with the upstream toe was provided by chemical grouting of the underlying strata.

Some improvement in this method was made by diluting the silicate with ammonia or with carbon dioxide to retard the time of set. However, the latest development in chemical grouting dates back only five years and consists of the simultaneous injection of silicate and reagents. By reducing the pH content of silicate to a certain carefully predetermined amount, it is possible to control the time of set from ½ min. to several hours.

By this method, developed by Charles Langer, a French engineer, it is possible to mix the chemicals before injection and inject them simultaneously. It also permits the prediction in advance of proper proportions and amounts to be used and reduces the time required for operation. This method eliminates the danger of silica-gel forming about the end of the pipe thereby preventing further dispersion of the chemicals through the pervious material. Three chemicals are employed in this process, sodium silicate, an acid and a heavy metal salt. Analysis of the soil to be treated reveals the natural salt content and defines the proportioning of chemicals. Knowing the permeability of soil and the pressure to be used, it is easy to compute how far the chemical will penetrate before it sets and therefore how far apart the grout pipes should be spaced. After a thorough investigation with all the modern means of soil mechanics, a definite grouting plan can therefore be made.

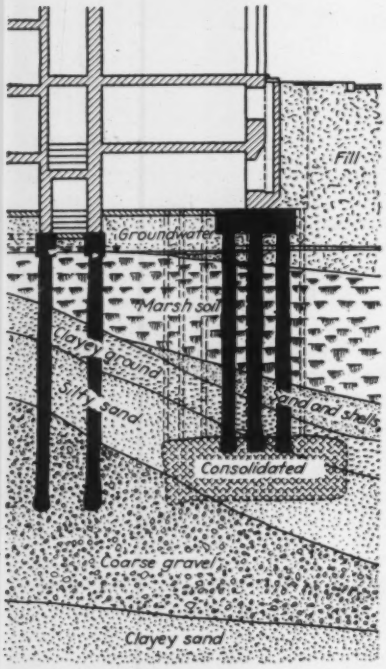

Fig. 3. Settling pile foundations of a Berlin apartment house were repaired by chemically grouting a mass of earth to form a solid footing for the piles.

Examples of practice

In the last fifteen years grouting with chemicals has been used on over 250 operations, most of them in Europe and some of them very large. The following are a few examples:

Stopping of Seepage—A ship canal parallels the Dordogne River at Agen, France. A pervious stratum, about 18 ft. thick and 20 ft. below the surface, lies between the river and the canal. The river has a relatively large water level variation while a constant depth is required in the ship canal. To prevent the variation from affecting the canal it was necessary to provide a cutoff in the waterbearing stratum. This was accomplished by chemically grouting a portion of the stratum, reducing the seepage from 40 gpm. to 0.15 gpm. per lin. ft.

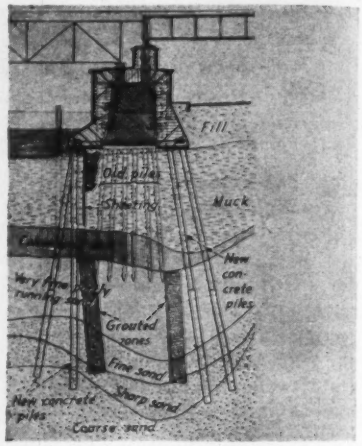

Fig. 4. In strengthening a railway bridge pier near Bremen lateral movement of the sand under the old piles was checked by boxing it in by walls of chemically grouted soil.

Improvement of Strata—The foundation material of the large rockfill dam in Bou-Hanifia, Algeria, was in part a finely fissured, dried-out shale. The fine fissures could not be filled with cement grout. They were grouted with chemicals, requiring 15,500 tons of silicate.

Increasing Bearing of Existing Piles—The apartment house (Fig. 3) in Berlin was settling. A solid mat, 6 ft. thick and 60 ft. long, was consolidated under the piles. This work was done 60 ft. under groundwater.

Preventing Movement of Quicksand—When the railroad bridge near Bremen, Germany, was rebuilt to handle increased loads, it was necessary to strengthen and enlarge the existing piers, which rested on 35-ft. piles. By enclosing the quicksand under these piles with walls of chemically consolidated sand (Fig. 4), the lateral motion of the sand enclosed was prevented, thereby increasing its bearing capacity to 1,300 lb. per sq. in. Fewer additional piles were required and no interruption to traffic occurred.

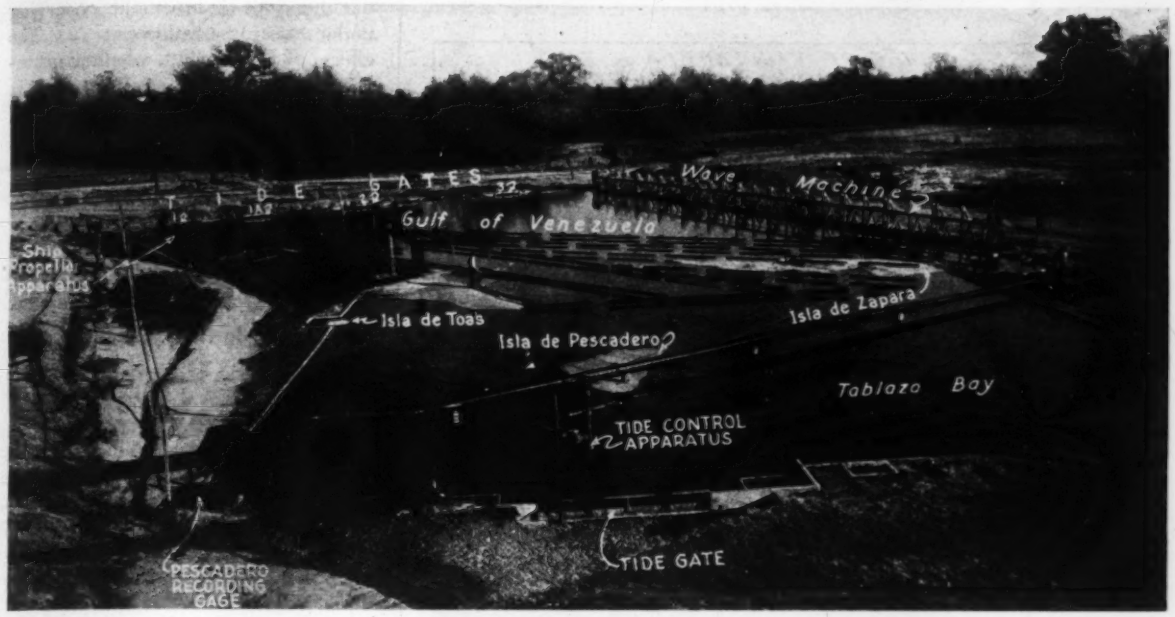

FIG. I. MODEL OF MARACAIBO HARBOR OUTLET TO DETERMINE CHANNEL THROUGH OUTER BAR.

Model Tests Solve Harbor Problem

PAUL W. THOMPSON

Director, U. S. Waterways Experiment Station, Vicksburg, Miss.

Contents in Brief—Three-year experiments with small-scale model of Maracaibo Harbor at U. S. Waterways Experiment Station furnish key to successful solution of maintaining a better bar crossing.

OIL SHIPMENT from Lake Maracaibo, Venezuela, has for some years been seriously hampered by the unsatisfactory condition of the outer bar across the gulf entrance to the outflow channel of the lake. With a view to deciding on possible improvement of the channel across this bar, model tests were carried out at the U. S. Waterways Experiment Station at Vicksburg, Miss., during 1936–38 for the Department of Public Works of the Venezuelan government. The tests pointed to the possibility of maintaining a more favorably located channel through the bar, and such a channel, excavated last year, has proved stable and satisfactory to date. The methods of model testing and analysis of this complex

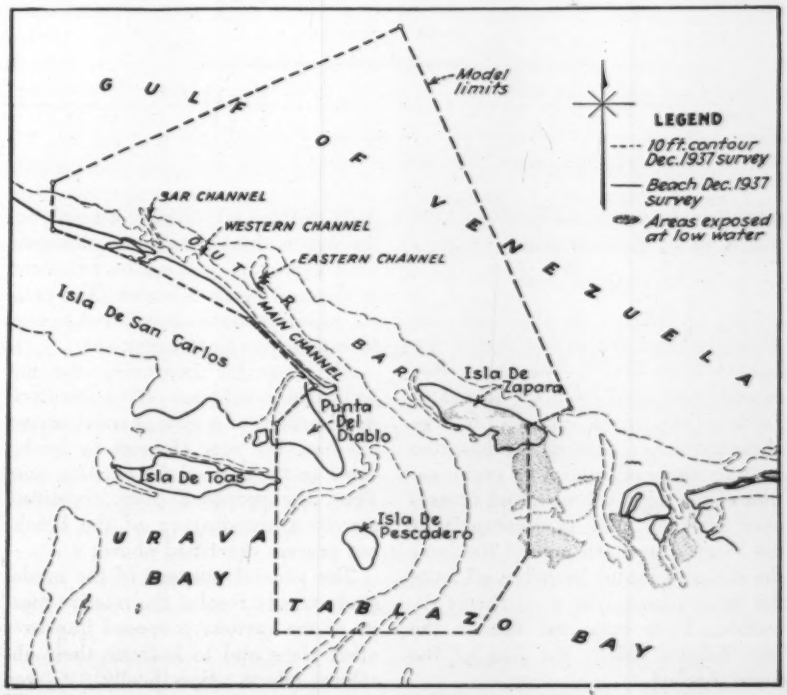

Fig. 2. Outlet to Maracaibo harbor, outer bar and channels to open water as of 1937 reproduced to determine most stable channel location across the bar.

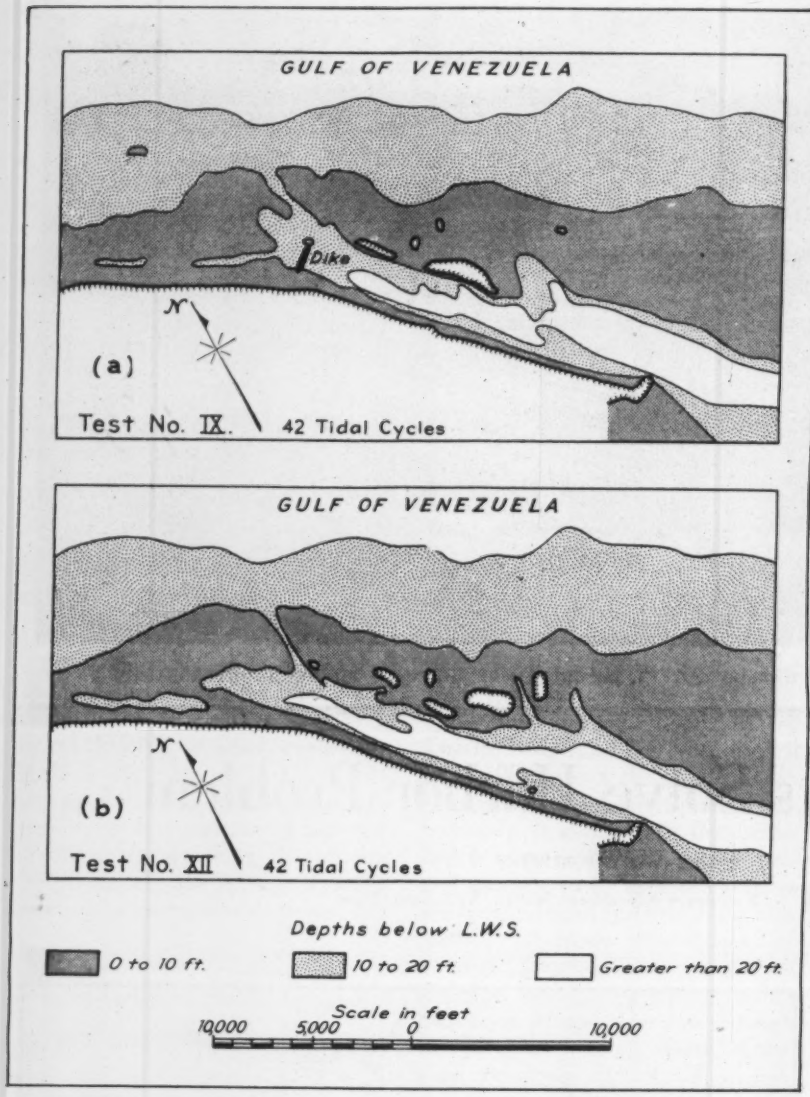

Fig. 3. The two most favorable plans of twelve by model; plan *b* of test No. XII carried out in 1938 has proved successful to date.

tidal problem are outlined by the following brief account.

Purpose of tests

The conditions at the Gulf entrance to the Lake of Maracaibo are indicated by Fig. 2. The Outer Bar extends westward from the Island of Zapara; the main channel follows along the inner edge of the bar; the western channel follows in prolongation of the main channel, and crosses over into the Gulf of Venezuela at the extreme western end of the bar; the eastern channel branches off from the main channel in a northerly direction. Flow into and out of the lake follows chiefly the line of the main channel.

The development of the bar is believed to be cyclical: the bar extends itself to the west; finally, a break-out through a channel to the east occurs, whereupon the process of extension to the west again is begun. The present eastern channel is believed to represent an incipient break-out.

All plans for improving the unfavorable conditions have involved development of a new channel across the bar, the new channel to be located to the east of the existing one. Thus, the proposed plans constitute merely a speeding-up of the break-out process described above.

The general purpose of the model study was to resolve the relative merits of the various proposed improvement plans and to indicate their absolute effects. Specifically, it was proposed to obtain from the model study information as to: (1) The likelihood of a break-out occurring under existing conditions; (2) The effects of a break-out, whether occurring under present conditions or as a result of improvement works; and (3) the most advantageous location for a break-out, and the most advantageous means of effecting the break-out. It was realized that the model study could be expected to satisfy its purposes in a qualitative sense only.

Design of model

On the basis of all data available as to tide, littoral currents, waves, winds and the effect of propeller action of ships, it was decided to include in the model the area immediately adjacent to the outer bar (Fig. 2), using loose material (movable bed) only on and directly along the outer bar. The scale ratios of linear dimensions, chosen in the light of earlier model experience and the analyses of the active forces in the present case, were 1:300 horizontal and 1:50 vertical. The model is shown by Fig. 1.

In the model it was necessary to provide forces which would combine in such a way as to approximate the effects of the resultant of the corresponding actual forces in the prototype. This involved consideration not only of the magnitude of forces, but also of the times and durations of action of the forces and rational methods of arriving at such determinations of forces through a process of trial-and-error (such process being known at the experiment station as "model verification"). The criterion for the adjustment process was provided by changes in bar configurations known to have occurred in the prototype. In this connection, detailed surveys for the outer bar were available for the years 1930, 1932, 1935, and 1937. The problem was to develop in the model that combination of strengths and durations of tides, littoral currents, propeller action, and wave action, which reproduced in the proper sequence the changes in bar configuration represented by the surveys just listed.

Confidence in the trustworthiness of the model results was based chiefly on the adjustment process just described. In its adjusted state, the model was believed capable of satisfying the purpose (enunciated in a paragraph above) for which the model had been designed. It was

realized that the model was not capable of indicating accurately absolute values as to time.

Tests indicate plan

The tests proper included the analyses of twelve different plans. Among the variables distinguishing the plans one from the other were: The use of a ballasted tanker in developing new channels; use of dredge; use of sills and groins; location and alignment of new channels through the bar.

In none of the tests was there an unaided break-out of a new channel. It is to be noted that none of the channels tested in the model (including the western channel) was able to maintain itself. In each case maintenance of the channel depended upon the action of propellers or dredges in loosening the bottom material.

Of all the plans tested No. IX shown by Fig. 3*a* was found most advantageous but it involved the construction of a sill—an operation which would be difficult and expensive. Fig. 3*b* represents the results of the plan involved in Test No. XII. The channel developed by this plan is only slightly inferior to the one developed by the plan of Test No. IX. All things considered, the plan of Test No. XII appeared to be the most practicable of all those tested. It was the plan selected for installation in the field.

Works approximating those involved in Test No. XII were installed in the field late in 1938—soon after the model study had been completed. All traffic has been shifted from the old (western) to the new (eastern) channel. Development and maintenance of the new channels have been aided by the operations of a hopper dredge.

Verification surveys obtained periodically from the field have provided indications as follows: (1) The western channel has shoaled completely, approximately as indicated by the model study; and (2) the eastern channel has been maintained but, as indicated by the model study, has shown no signs of rapid and large-scale development. As indicated by the model study, the new eastern channel as installed in the field results in conditions which are more economical and less hazardous than those which existed in the old channel through the bar, and navigation has thus been improved.

Sacramento Resells Land When Unpaid Taxes Accrue

WHEN BUSINESS CONDITIONS were pretty bad in 1933, Sacramento, Calif., suddenly found itself in the real estate business. The emergency arose when a new real estate subdivision, in which sewers, lights, pavements, sidewalks, etc., had all been put in, simply failed to attract buyers and the subdivider quit. The city was left with a $350,000 investment and 476 unsold lots on its hands. Not only was this problem satisfactorily solved by the addition of a realty sales department to the municipal organization, but the department has been made permanent and is materially helping the city to compensate for the limited municipal income resultant from the fact that 40 per cent of the area that makes up the state's capital city is exempt from taxation.

Although somewhat different methods had to be used in the large tract on which the new department started its activities, in general the same principles have been applied to individual lot sales. Altogether the new department has sold 1,100 lots to date, 500 more are available for sale, and next fall, at the expiration of a three-year moratorium on general taxes, some 1,500 more lots probably will be taken over for resale.

When general taxes go unpaid for five years the property reverts to the state; on public improvement delinquencies, however, the legal limit is one year. Sacramento has adopted the policy of contacting the owner after improvement taxes are delinquent two years. The rate at which the delinquency is eating up the owner's equity is explained and almost invariably the owner is willing to make some arrangement with the city whereby he signs a quit-claim deed. If this cannot be arranged, legal proceedings are started at once.

Under legal procedure as followed in Sacramento, three weeks to two months are required to complete a title action. When the title has reverted to the state the city acquires it by paying off the accrued county taxes whereupon title vests in the city. Charged to the account of each lot are the total taxes, the cost of legal procedure (which is rarely more than 5 per cent of the price), plus the agent's commission if the land is sold through a real estate broker. This commission is 6 per cent of the sale price with a minimum fee of $50 per transaction.

In the 1,100 lots thus far sold 95 per cent of the owners to whom the matter was fully explained gave quit-claim deeds: the remaining 5 per cent were mostly non-residents or persons whose present location was unknown. In all of the city's transactions where legal action was necessary there has not been a single contest of the procedure transferring title to a new owner.

In the initial venture some five years ago when an entire tract had to be handled, methods were much as in the usual subdivision plan. Building restrictions were set at $5,000, but by reason of the lower price at which the lots were offered, all 476 were sold within 2½ years.

In the resale of individual lots the city's procedure is first to acquire title and then to advertise the property and list it with real estate dealers. If the original owner desires to reclaim it he is told at what price the city can sell, and if he makes a bid of that amount, preference over other bids is given.

In advising other cities, from which inquiries have come, as to the success of this plan, George B. Welch, manager of Sacramento's realty sales department since its inception, has suggested that the first step is to get the cooperation of real estate brokers. This can be done by assuring them that the city will not disrupt the market price and will pay the usual commission on sales, thus really increasing their volume of business and protecting their interests. The remainder is simply business-like management; psychology favors the city because purchasers feel they are getting bedrock prices—and they are.

Why So Few Famous Engineers Today?

FARLEY GANNETT
Harrisburg, Pa.

Contents in Brief—In a thought-provoking discussion, Farley Gannett asks why so few engineers of today have attained fame comparable to that of many engineers of past decades and puts forward several of the most plausible explanations that have been suggested to him.

THE DEATHS of Harrison P. Eddy, Hugh L. Cooper and Guglielmo Marconi left notable vacancies in the fast dwindling ranks of the world's famous engineers. Within a decade Allen Hazen, General Goethals, Leonard Metcalf, Thomas Edison, James H. Fuertes, Frederick P. Stearns, Waldo Smith, Charles P. Steinmetz, M. I. Pupin, John R. Freeman and others have departed, leaving monuments in waterworks, bridges, sanitary and electrical engineering works. It was through these men that engineering developed in the last two or three decades, and it was by reason of men like them and Rudolph Hering, Alfred Noble, the elder Fuertes, Waring, George Westinghouse, Desmond Fitzgerald, Barclay Parsons, Roebling, Eads and others that engineering became an important and respected profession in the United States; their names became and have remained familiar and honored among members of their profession, some of them also to the general public.

A few engineers of note are still with us, such as Ralph Modjeski, John F. Stevens, John Hays Hammond, Herbert Hoover and Arthur Morgan. We all know these men by reputation. There are not many that could be added to this list today. Ten years ago there were a score or more of names well known in engineering for years; only a few have made their reputations recently. The list of great ones has been steadily declining, and no recruits seem to be filling the depleting ranks.

The editor and I were discussing this subject one day and he suggested that I sharpen my pencil on the question of fame or greatness of engineers today compared with a generation or so ago. We agreed that there were few engineers of recognized fame today compared to the large number of the past. We considered whether ability alone made fame, and finally seemed to agree that there are as many and perhaps more able engineers today than in the past, but many able engineers never become famous, and conversely, some famous engineers perhaps could not be considered so very able.

Sources of recognition

There are many reasons why an engineer becomes well known; it may be ability, it may be luck, it may be character, it may be salesmanship or ambition or perseverence or any number of other things, or a combination of several. Without some ability it would be hard to think of an engineer becoming famous, but a combination of ability and a decided character would be a great help. Add salesmanship or showmanship and a little luck and a lot of perseverance, and maybe there you have it. But there must be daring and courage because these make for human interest, and there is seldom fame without human interest.

One cannot pass through the Panama Canal without thinking of Goethals, nor visit the sources of New York's water supply without Waldo Smith's name coming to mind, nor cross the Brooklyn Bridge without thinking of Roebling. Water filtration means Hazen; sewage purification, Waring, Hering and Eddy; bridges mean Eads, Modjeski and Roebling; electricity and dams mean Edison, Steinmetz and Cooper. Did these men's deeds make them famous or the men themselves make great their deeds? These men and their accomplishments brought the attention of the world to engineering as the profession which forwarded not only the development of our nation through railroads, water-power, steam and electric power, but also our health and comfort through water purification, sewage disposal, etc.

We naturally are more appreciative of and familiar with the great names of our own branch of engineering. If names have been omitted that should have been included, it is for that reason.

What attracts public notice?

Why were the men named above widely known? Why are their names and their deeds living after they are gone? Did these men achieve fame because they were among the ablest of engineers, or were some of them mediocre engineers who had something else that made them prominent among others, that caused them to be remembered and their names to become familiar to succeeding generations of engineers? Was Hugh Cooper known all over the world as a great hydro-electric engineer because he built the Keokuk, Dnieper and Muscle Shoals dams, or was it because he brought to his work a boldness which others lacked? Was he a great person as well as a great engineer? Would he have been equally notable had he been a lawyer or doctor? Was George F. Swain only an educator and a bridge engineer, or was he such a powerful personality that his presence, as well as his ideas, pervaded the classroom, the college halls and dormitories and directors' meetings? Allen Hazen was a student, a thinker and an experimenter. He worked in the then new field of water purification, and it was his experimenting and testing and inquiring that gave us definite knowledge that waterborne disease could be almost eradicated in cities by filtering the water. He developed a new channel to longevity. He was a doctor who, like Victor Heiser, devoted his life to preventive measures.

The consensus of opinion among

men with whom I have talked seems to be that it requires a pioneer to make a name for himself, and they seem to think that the possibilities for pioneering in engineering have ceased. But have they? Will pioneering ever really cease? New developments, changes in old, involve pioneering. Is it impossible for an utterly new method of water or sewage purification to be developed? There are so many things being done on a far larger scale that it is hard to tell what is pioneering and what is not, but it just as unfair to say that there is no more pioneering possible in engineering today and in the future as it would have been to say it 50 years ago.

If Hugh Cooper pioneered when he built Keokuk Dam, Muscle Shoals Dam or the dam and powerhouse on the Dnieper River in Russia, certainly the man who is responsible for Boulder Dam is a "bolder" pioneer. But who knows who conceived Boulder Dam? It might be said that Boulder Dam was simply larger than other dams, but on the other hand, General Goethals achieved fame by building the Panama Canal and this was only a step beyond the Suez Canal and other lesser canals built in many parts of the world. Furthermore, Goethals was a government employee, just as were the men who conceived and built Boulder Dam; but Goethals became a famous man, whose name is familiar not only to our own profession, but to educated people over the world.

Couldn't we call the Golden Gate and San Francisco Bay bridges pioneering? There is nothing like them in the world in size, nor in difficulty or cost; yet the men who conceived and built these bridges have not become world-renowned as did Eads and Roebling—and there were many bridges prior to their day.

Another answer given me was that much of the great work today that would otherwise make men famous is being done by the government and that names and individuals are submerged in government work. Nevertheless, there are hundreds and perhaps thousands of engineers in private practice today who are not doing government work and none are even projecting their heads above the horizon. Another suggestion is that engineers who are not working for the government are mostly working for corporations and that corporate employment eliminates the possibility of a man's making a name for himself. He may make his corporation famous, but not himself. Steinmetz worked for a great corporation and yet he achieved fame.

JAMES BRINDLEY 1716-1772

British canal builder, one of the great pioneers of civil engineering who won fame through the boldness of his designs.

Another suggestion is that for every engineer graduated from engineering schools fifty years ago there are from 10 to 100 being graduated now; that it is harder to emerge from a large field than it is from a small one. Also, where competition is so strenuous as it is today and has been for twenty years, one gets much less opportunity for accomplishing great results.

The engineering profession is being used for greater things today than ever before in this or any other country's history: the TVA, the Columbia River developments, Boulder Dam, the great bridges at San Francisco and New York, flood control, housing, New York's and other great cities' new water supplies and sewage disposal on a scale not dreamed of previously. Why are not personalities being developed from these great works, why are not engineers being made famous thereby? This should be a time not only of greatness of our profession but for the individuals in it.

Probably more people the world over knew Herbert Hoover than any one else who ever was a member of our profession. They didn't know him while he was practicing his profession, however. His fame does not rest on his engineering works, but his ability and training obtained while he was so engaged perhaps enabled him to do the things which brought him fame. General Robert E. Lee was first an engineer officer. Stone and Webster are widely known names of engineers, but had they not gone into finance their engineering works might not have made them prominent.

There is an old saying that you can't keep a good man down, and maybe that is what it is—that there aren't really good men developing in the profession today. And by a good man, I mean not only intelligent, educated, trained and experienced, but I mean a man of character, energy and above all, enthusiasm and a certain amount of showmanship.

Such a man should be able to rise from those around him to get above the surface and have his name resounding from the housetops whether he works for Uncle Sam, for a great corporation or for himself.

Earthquake Resistant Design

Four A.S.C.E. convention papers relate to progress and review design features used on three notable West Coast bridges

A REVIEW of practice and progress in earthquake-resistant design constituted the program of the structural division at the convention of the American Society of Civil Engineers in San Francisco, July 26–28. In the report of the convention published in our Aug. 3 issue, p. 164, it was noted that the structural division papers were exceptionally significant, and would be reserved for separate abstract. The four papers related to general progress in the art and to design methods used on the San Francisco Bay, Golden Gate and Pit River bridges. The abstracts follow.

PRESENT KNOWLEDGE

Structural design progress in the field of earthquake-resistant construction has been divided into successive periods, each usually initiated by a major quake which greatly increased incentive to research. Defective workmanship exposed by the Long Beach earthquake created a profound impression when it appeared that, had the quake occurred during school hours, thousands of children would have been injured or killed. A direct result of this public awakening was that good workmanship is now the rule rather than the exception, and special emphasis has been placed upon making the engineer responsible for both design and supervision of construction.

California and Japan have been fertile fields for observation and development of improved design, and the engineers who have had repeated experience with major earthquakes have pooled observations and conclusions into a code (just printed) representing 13 years of collaborative study. Precedent to this definite action have been shaking table experiments at Stanford and California Institute of Technology, seismological field surveys by the U. S. Coast & Geodetic Survey, and the enactment in California of a state law (the Riley Act), which was a direct outgrowth of the Long Beach earthquake, requiring all buildings (except farm buildings, and like structures) to be designed for a lateral force of 2 per cent of the vertical load, or a wind pressure of 20 lb. per sq. ft. from any direction.

The following is what we think we know at the present time about earthquake-resistant construction.

(1) Foundations should be secure against settlement, and all the following statements assume stability of foundations.

(2) The ground characteristics of the general area are of great importance in evaluating probable earthquake forces on a structure—different areas may have different characteristic periods of vibration.

(3) In buildings with floors sufficiently stiff to act as distributing plates, the distribution of lateral force to the various structural elements is in proportion to their rigidities. The walls of a building furnish the major resistance.

(4) All vertical structural elements resist torsional moments in proportion to the square of their distances from the centroid of resistance, and also in proportion to their rigidities.

(5) The centroid of resistance of the vertical structural elements should coincide with the centroid of mass of the building, or else the torsional moment must be provided for.

(6) Buildings less than 100 ft. high, whose height is not greater than twice their least base dimension, may be designed as rigid structures for a lateral force of 0.1 *g.*, using ordinary working stresses and with reasonable assurance of no earthquake damage. With increased working stresses, say as high as 1½ times the usual working stresses, the damage will probably be minor.

In all buildings, shear distortions are of much more importance than flexural distortions.

(8) In addition to the fundamental period of vibration, the second and third modes may affect the structure, although to a lesser degree.

(9) Increase in flexibility of the basement story, or the first story, results in decreased dynamic shears in the stories above, and offers a possible method of earthquake design.

(10) Vibration experiments with models of buildings with structural steel frames and brick masonry walls, 16 to 29 stories in height, justify the tentative conclusions that tall office buildings must be regarded as flexible rather than rigid structures, and that they should be designed for dynamic shears. From these experiments it appears that the horizontal shear varies from story to story. Considering only the fundamental period of the building, and for the buildings studied, the story shears may be assumed to increase from 1/20th *g.* in the first story, to 1/7th *g.* in the top story, the mass in each case being taken as that above the floor in question. In buildings with flexible first stories, these coefficients may be reduced.—HENRY D. DEWELL, *Consulting Engineer, San Francisco.*

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BAY BRIDGE DESIGN

The protection afforded to the San Francisco-Oakland Bay Bridge by making definite provision for earthquake resistance added less than 5 per cent to the total cost of the structure. Basic assumptions made were that the bridge might have to resist ground motion with horizontal acceleration of 0.1 *g.*, with a period of 1½ sec. and a corresponding amplitude of 2.2 in. This is a longer period than is probable but was chosen because, with other assumptions used in the design, it generally produced higher stresses than the same accelera-

tion with a shorter period. It is anticipated that the period in the destructive phase of the earthquake is not likely to exceed 0.7 sec. Although after-shocks of the general order of 6 to 8 sec. may be expected, there is no record that such shocks cause structural damage.

In analyzing effects on piers it was assumed that a pier, when moved by the rock on which it rests, will displace the surrounding water and, possibly, part of the surrounding silt. Westergaard's method of dam analysis was applied to this problem and, because water is present on both sides of the pier, the acting forces were doubled.

In an earthquake of transverse direction, earthquake stresses in the towers are caused by inertia in the suspended structure as well as in the towers themselves. Natural periods of the stiffening trusses in the center spans vary from 6.1 to 18.5 sec., and in the side spans from 1.6 to 4.9 sec. The suspension system, as a whole, has many characteristics of a steel spring. The natural period of the unloaded suspended structure has been found by measurement to be 6.2 sec., while it is 7.3 sec., when fully loaded.

The flexibility of the suspended system is indicated by the fact that the center of the longer span has a calculated range of vertical motion of 25 ft.; the top of tower 2 has a horizontal motion of 6½ ft., and in a wind of 60 mph the center spans will have a horizontal deflection of 9 ft. Stresses, especially in the suspended structure, are controlled by these distortions. Compared therewith, the amplitudes of any earthquake motion and their accompanying stresses would be relatively small.—NORMAN C. RABB and HOWARD C. WOOD, *California Toll Bridge Authority, San Francisco.*

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GOLDEN GATE BRIDGE

Analysis of earthquake effects on the Golden Gate Bridge gave assurance that for an assumed horizontal acceleration of 7½ per cent of gravity, resultant stresses on the substructure will be relatively small, and on the superstructure will be less than the effect of a 30-lb. wind load.

To get earthquake movement into computable form, it is assumed that the earthquake will approximate simple, harmonic motion. Then a simple equation for maximum acceleration can be written as follows:

$$a = \frac{16A}{T^2} \text{ in which}$$

a = maximum acceleration in unit of length per sec. per sec.

A = amplitude in unit of length.

T = period of time for full vibration in sec.

Destructive earthquakes do not involve large displacements: amplitudes seldom exceed 1.5 in. The lowest period of vibration observed is about 1.35 sec. With these data in mind an amplitude of 1.24 in. and a period of 1.3 sec. were assumed. For these values a maximum acceleration of 2.42 ft. per sec. per sec. resulted, or about 7.5 per cent of gravity.

For the substructure (piers and anchorages) in which there will be no elastic deformation, the horizontal force of the earthquake was assumed at 10 per cent of g. The shock was assumed to come from any direction. On the south pier, which is in deep water, the effect of the water was also included in the calculations. With these assumptions, increased pressure due to earthquake on the south pier would be 2.5 tons per sq. ft., and on the Marin pier, 3 tons per sq. ft. Highest shears would not be more than 10 lb. per sq. in.

The rigidity assumed for the substructure does not hold for the superstructure, which is flexible to a high degree. Longitudinal earth shocks tend to increase the horizontal cable pull on one side of a tower and to decrease it on the other. However, all such effects were found to be greatly decreased because the acceleration would be absorbed in the deflection of the tower. The suspended center span, with an expansion joint at each end, can move longitudinally, and thus its weight cannot be accelerated by earthquake. In general, the flexibility of suspension bridges is of great assistance in resisting earthquakes.—LEON S. MOISSEIFF, *Consulting Engineer, New York, N. Y.*

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TALL BRIDGE PIERS

A double deck steel cantilever bridge on piers of 360-ft. maximum height and of over 300-ft. submerged height will be required to carry a railroad and a highway across the Pit River on a relocation around Shasta Reservoir, under construction in northern California. To determine earthquakes effects on the structure three fundamental investigations were undertaken.

(1) The effect of submergence on the apparent or virtual mass of a pier.

(2) Earthquake effects in a bridge on elastic piers fixed at the base.

(3) The stability of a rigid pier rotating at the base.

These investigations have led to new conceptions of earthquake effect and to better design methods. They explain why piers and similar structures, although not originally designed to withstand earthquakes, are often little affected by them. They show that if actual recorded earthquake acceleration intensities are applied as a steady force to the Pit River Bridge piers, the resultant generally will fall outside the base, indicating failure. However, if this assumption were correct, many bridge piers and similar structures in the vicinity of the Long Beach, Cal., earthquake of March 10, 1933 and of the Helena, Mont., earthquake of October 31, 1935, would have failed, which was not the case.

Mathematical analyses supplemented by shaking table investigations lead to the conclusion that any lateral earth movement tending to tip the pier reverses its direction before the mass of the pier can follow such movement, and that this restoring effect takes place as soon as the lateral movement is of sufficient magnitude to throw the resultant force toward the edge of the base. Under these conditions the pier no longer behaves like an elastic structure fixed at the base. On the contrary, it acts as a rigid structure rotating at the base.

The design criteria that resulted from these studies can be summarized as follows for the Pit River Bridge:

(1) Design the pier base with strength and stability for the usual lateral loads, such as wind and traction forces, without any consideration of earthquake effects.

(2) Design the pier above the base for lateral earthquake forces of such intensity that the resultant is at the edge of the base.

While these design criteria are for the piers only, the investigations were extended to include the superstructure in its entirety.—J. L. SAVAGE, *Chief Designing Engineer, U. S. Bureau of Reclamation, Denver.*

Old Slide Compels Radical Remedy

G. H. ALLEN
District Engineer, Indiana Highway Department
Vincennes, Ind.

Contents in Brief—Old slip on sidehill fill becomes active, destroying system of steel baffle-plate slope retainers; concrete retaining wall structure found to cost less than fill replacement.

A SLIP in a sidehill fill that was finally checked by a system of steel baffles, which held the ground for three years, was described in *Engineering News-Record*, July 1, 1937. Now the writer has to tell of the recurrence of the slip, the destruction of the steel baffle system and the redesign of the road to insure stability regardless of future fill movement. The new road is shown by Fig. 1.

The road, as rebuilt in 1934 and as it remained until May 15, 1937, is shown in Fig. 2, top left. The surface was oil neat, and in 1937 it was decided to replace it by a higher type. To keep excavation to a minimum the survey centerline was run about 18 ft. from the south shoulder line at the slide. After the plans had been started a small crack appeared at the line of the old slide. On May 15, 1937, the shoulder dropped 2 ft. and an investigation was started to determine what had caused the slide and the method necessary for correction.

Twelve borings were made just below the slide crack along the shoulder and four additional borings were made down the slope. These borings disclosed that the material in the slide was alternate layers of shale, fire clay and sandstone. The lowest plane of sliding was 15 to 20 ft. below the surface, and there was sufficient fire clay at other points to allow sliding in other planes.

The plan of the slide in Fig. 2 shows the relative position of the road and railroad at a point adjacent to the slide. At the slide the movement of earth was within 20 ft. of the centerline of the railroad, and because the railroad is on a fill 45 ft. above the river it was probable that the slide would, if constantly loaded, flow over the railroad and down to the river, or force the railroad fill down the slope. It was, therefore, necessary either that the slide be stopped or the road be built so as to eliminate the necessity of adding earth to the shoulder.

The field investigation also disclosed that a clay mine had operated adjacent to the slide between 1830 and 1850. The only visible evidence of the mine on the surface was a steeper slope where the entry had caved in. The old settler who told about the mine also related a story he had been told by men working there; they said a pond of water formed in the mine near the entry and a boat was used to get to the clay workings. In February, 1937, the

Fig 1. Retaining wall set on rock isolates road loads from hillside fill, which had a tendency to slide toward river.

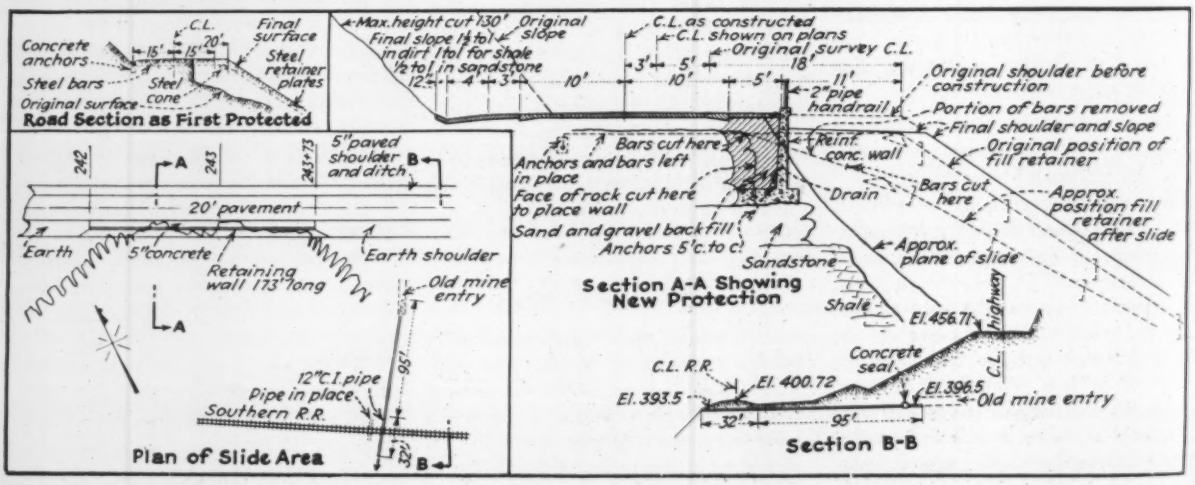


Fig. 2. Slip in a hillside fill and new road construction that avoids further hazard of sliding ground.

Ohio River rose about 50 ft. and water covered the railroad. It was believed that some water had seeped into the mine besides saturating the toe of the slide. As the steel baffles extended down only 45 ft. from the top of slope it is possible that the toe slid away and allowed sliding to take place under the baffle plates.

Relocation best resort

The borings which were made on the slope showed that the sandstone and shale ledges stepped out (Fig. 2, Sect. AA) and to provide subsurface drainage it would be necessary to place a series of pipes to catch all the possible seepage between ledges. It was also believed that to prevent sliding it would be necessary to replace the conglomerate mixture in the slide with suitable fill material, and as this involved 15,000 cu. yd. of excavation and a like amount of special fill it was considered to be too expensive. The design department was then asked to move the road centerline 5 ft. north with the idea that some method of correction could be worked out at the time of construction.

The contract for the new road was let and as the excavation of the cut proceeded it was found possible to gain another 3 ft. at the ditch line; also because the slide area was on a vertical curve it was possible to shift the road another 3 ft. without affecting the appearance of the alignment. After the rough grade was complete the face of the rock was exposed the full length of the slide area as shown in Fig. 2, Sect. AA. The ledge exposed about 9 ft. below the pavement elevation was sandstone, considered a suitable foundation for a retaining wall, and the uneven ledges above were cut back as shown to place a continuous wall about 9 ft. high.

As the backfill behind the wall is a sand-gravel material which drains to a pipe placed on the footing, there is practically no hydrostatic pressure on the wall. Also there is very little surcharge load as the pavement has less than 2 ft. of cushion in most places and is entirely over the higher ledge. The wall prevents the sloughing of material from under the pavement, allows a 5-ft. paved shoulder, and places very little additional load on the sandstone ledge that supports it. Fig. 1 shows the completed job with handrail and paved shoulder adjacent to the wall; the pipe handrail was used because this location provides a fine view of the Ohio River, 100 ft. below, which would be partially obscured by a closed rail.

Mine water drained off

The only attempt made to correct the slide was to drain the old clay mine. After the entry was exposed it was found that the water level in the mine was about El. 399. The existing pipe under the railroad was too high to take care of the drainage so a 12-in. pipe was laid from the entry to a point on the opposite side of the railroad as shown in Fig. 2, Sect. BB. A concrete seal was placed at the mine entry to prevent as much seepage as possible into the toe of the slide. It is believed that by keeping the water level in the mine to El. 396.5 one source of saturation of the toe of the slide will be eliminated. The highway is at El. 456.7.

All investigational and engineering work was handled by engineers of the Vincennes district of the Indiana highway department; and construction was performed by the general contractor for grading and paving, The Calumet Paving Co., Indianapolis, Ind.

Precast Liners for Drainage Ditches

A PRECAST CONCRETE SLAB for lining ditches, which is easy to make and install, has been developed at Jacksonville, Fla., where it is used for mosquito control drainage. A ditch lined with these slabs requires practically no maintenance and the lining eliminates the possibility of standing pools of water. Furthermore, the cost is considerably less than linings heretofore employed.

As designed by H. D. Peters, sanitary engineer for the city, precast slabs can be used for lining both the bottom and the sides of the ditch, and they can be adapted to any depth of cut. Each slab is 4 ft. long and 2 in. thick. The sides are beveled so that one side measures 8 in. wide and the other 9½ in. in width. The bevel edge assures close-

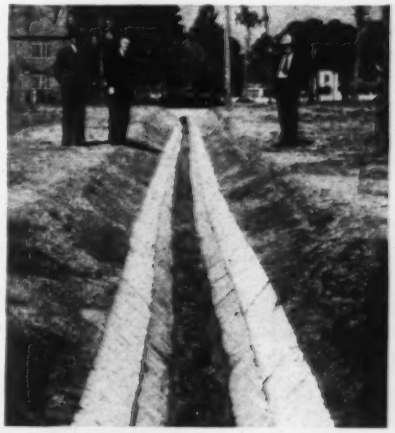


Fig. 1. Method of assembly of precast slabs (below), and a section of the completed lining (right).

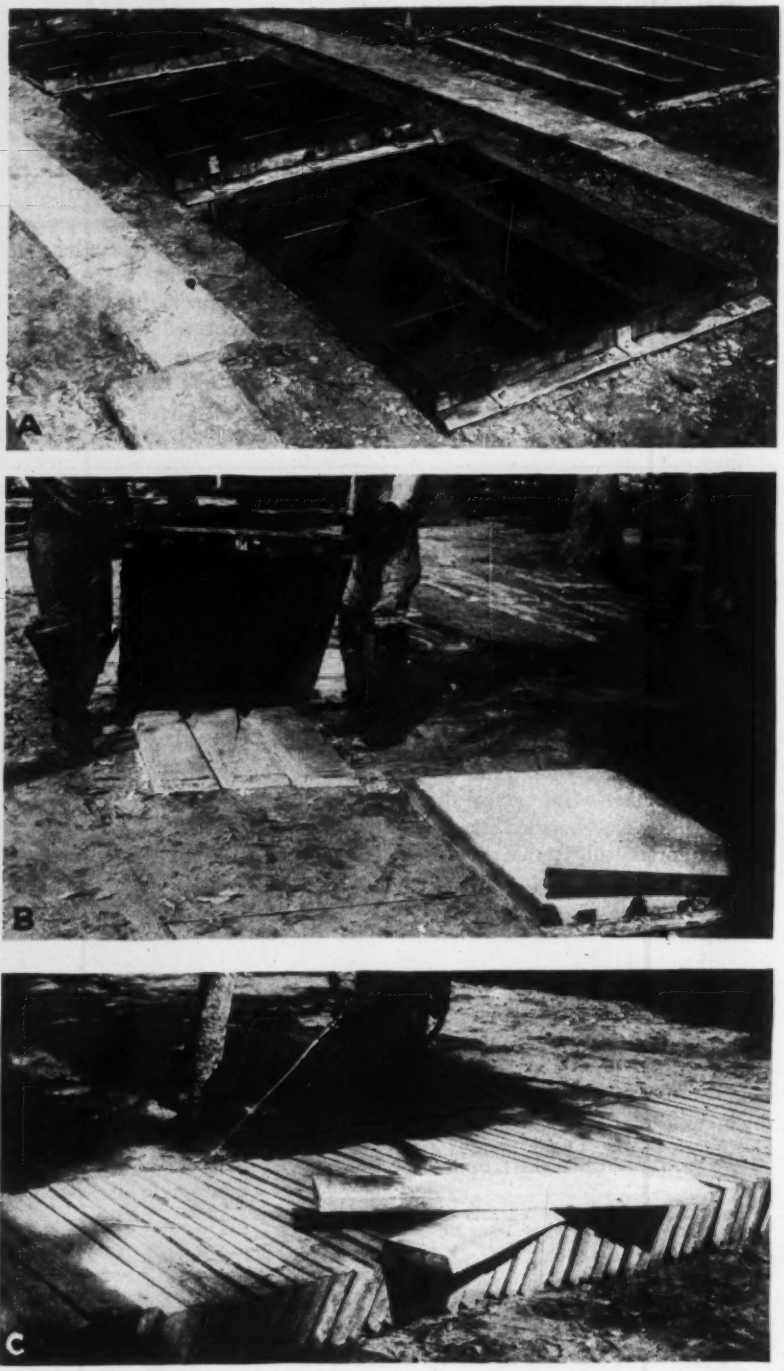


Fig. 2. How the precast slabs are made: (A) The wood forms in each of which three slabs are cast; note the ¼-in. rods for molding the wire hole; (B) to remove the hardened slab the form is turned upside down after an end piece is taken out; (C) finished slabs are put in a stockpile and wetted down for 7 days.

fitting joints between slabs; this discourages weed growth yet permits seepage of groundwater to relieve hydrostatic pressure. Transversely through the quarter points of the slabs, a ¼-in. hole is molded through which a wire can be passed to tie the slabs together.

In Fig. 1 is shown the method of assembly and the appearance of the finished lining. The 8-in. bottom width has been adopted because this will accommodate an 8-in. square-point shovel and thus simplify cleaning.

Forms for casting the slabs are made by building a box with sides 2 in. high, and an inside length of 4 ft. The width of the box is such that it will accommodate three slabs plus the space taken up by the partition (see Fig. 2A). The partition pieces have a ¾-in. bevel on each side, and the inner face of the long side pieces of the form are likewise beveled. Holes large enough to accommodate a ¼-in. rod are drilled through the sides and partitions at the quarter point of the slab.

It should be noted that the end pieces of the forms are not fastened permanently to the form but are held in place by three angle-iron braces. This construction makes it possible to slide the end pieces out of the form to facilitate removal of the finished slab. The manner in which the slabs are removed is shown in Fig. 2B.

The slabs are made with a 1:2½:3¼ concrete mix, using ¾-in. gravel. With the forms oiled and the ¼-in. rods in place, the concrete is poured and struck off level with the top of the forms. Striking the form with blows from a mallet helps to secure a dense concrete. When the concrete is stiffened slightly the surface is finished with a steel trowel. The ¼-in. rods are then withdrawn to leave a hole through the slab.

The slabs are taken from the forms after 18 to 24 hours, put into a stockpile and kept moist for at least seven days.

* * *

Long Island Groundwater Replenished by Rain

As a result of the September hurricane and other heavy rain storms in Long Island, groundwater levels in April, 1939, in various parts of Nassau and Suffolk counties were as high as, or higher than, at any time since observations were begun in 1933, according to David G. Thompson, senior geologist of the U. S. Geological Survey.

In western Suffolk County, Lake Ronkonkona, a huge open well about a mile in diameter the level of which reflects closely the groundwater level in the territory surrounding it, was within 2 ft. of the previously observed highest level, reached in 1891. Inasmuch as general observations go back for 60 or 70 years prior to 1891, the present high level is almost as high as it has been at any time in more than 100 years.

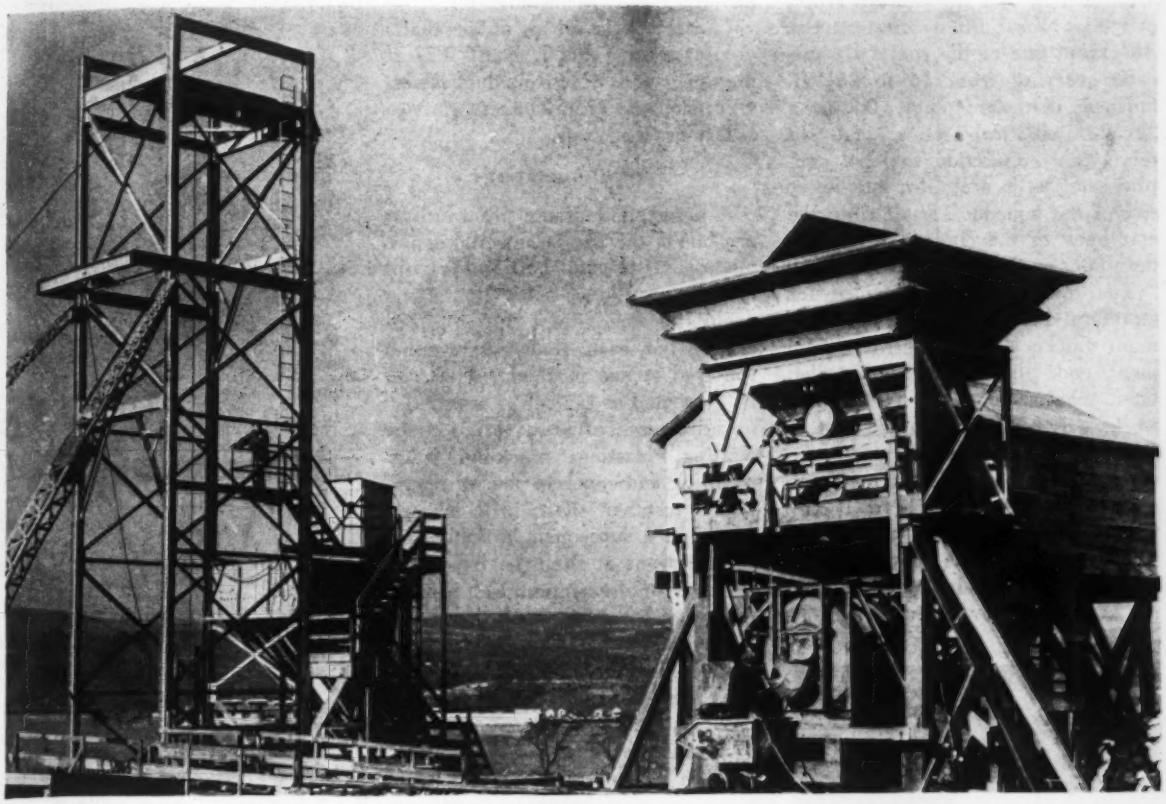


FIG. I. HEADFRAME AND CONCRETE PLANT FOR SINKING SHAFT SIX, DELAWARE RIVER AQUEDUCT.

Shaft Sinking on the Delaware Aqueduct

Contents in Brief—Modern shaft sinking procedure was demonstrated by three contractors in constructing 23 shafts aggregating 15,000 ft. in depth as part of the New York City water supply development. The shafts were sunk to various depths in different types of ground and under surface conditions varying from high-class residential areas to wild mountain locations. Drilling, mucking and concrete lining operations at one shaft described in detail as generally typical.

TO EXPEDITE start of construction of the new 85-mile Delaware River Aqueduct from the Catskill Mountains to New York City, the New York Board of Water Supply awarded contracts for sinking 23 of the 30 shafts required for the project in advance of letting tunnel contracts. These shafts, from 314 to 1,550 ft. deep, are now complete and tunnel driving has started under separate contracts. In constructing the shafts three contractors—Dravo Corp., Frazier-Davis Construction Co., and W. E. Callahan-J. P. Shirley Co.—holding the contracts for the work, demonstrated the latest art of shaft sinking under various conditions of ground, water infiltration, size, depth and location of hole. Location of the shafts varies from the residential districts of Westchester County to wild mountain territory in the Catskills.

The new aqueduct, all in tunnel and extending for 85 miles from the proposed Roundout Reservoir at Lackawack, N. Y., to Hill View Reservoir in Yonkers, requires 30 shafts, some for construction purposes alone to divide the tunnel into sections approximately 6 miles long, but mostly for waterway and control purposes as well as for construction. Separate contracts were awarded for construction of 23 of the 30 shafts; six shafts (Nos. 11 to 16, incl.) were subsequently let with corresponding tunnel contracts, and shaft 1, at the intake, is included in the contract for Lackawack Dam. The Delaware River project, which will increase the city's water supply by 50 per cent, has been described in *ENR*, Jan. 14, 1937, p. 41, and Mar. 31, 1934, p. 698.

Only six of the 23 shafts under the shaft contracts were sunk for tunnel driving purposes alone; the other seventeen are being used as construction shafts during the tunnel work and later will be equipped with control works for various operating purposes. Shafts 9, 10, 17, 18 and 19 are double shafts, close together but entirely separate, for connections to

West Branch and Kensico reservoirs and East View filters, respectively. All except one of the shafts are circular, varying from 14 to 26½ ft. finished diameter; shaft 23 is a 22x32-ft. elliptical section, and will serve as a connection to local supplies and as a drain for the lower end of the aqueduct. Shaft 6 on the east bank of the Hudson will drain the upper section.

The shaft contracts provided for structural lining only, 19½ in. minimum thickness; all control equipment and plugs will be installed later. Shafts not used for control works will be sealed top and bottom with concrete plugs. Embedded in the concrete lining the full depth of each shaft are ten service pipes for use by the subsequent tunnel contractors. These pipes, four 6-in., two 4-in., one 3-in., two 2-in. and one 1-in. lines, are being used for air, water, drainage and electrical installations.

Fig. 2. Going on shift at shaft 8. Note the trap door over shaft opening, kept closed except for passage of bucket; sliding dilley above bucket follows part way down shaft to keep cable from swinging.

Characteristics of all of the shafts are given in Table I. Shaft 2 is at the upper end, near Roundout Reservoir; shaft 23 is in Yonkers, next to Hill View Reservoir.

Typical operations

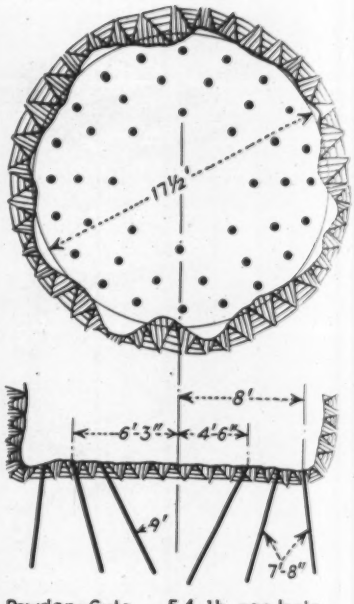
A description of the procedure at any one of the 23 shafts would depict modern, up-to-date shaft sinking operations, and since space limitations prohibit a detailed description of operations at each shaft, the routine followed at one particular shaft can be presented as typical of operations on all three contracts. Variations of drilling, blasting, mucking, water handling and concrete placing operations on other shafts will be presented in a subsequent article.

Operations at shaft 2A have been selected for description, not because they were better or worse than those at any other shaft, but because this shaft is not only the deepest on the line, but is one of the deepest ever sunk except for mining purposes, and is therefore the most spectacular. William Fowler was superintendent in charge of the shaft under Arthur C. Dennis, general superintendent, for Dravo on shafts 2, 2A and 3.

Shaft 2A lies in a remote valley high in the Shawangunk Range of the Catskills. Its top is at El. 1145; its bottom at El. —405. To provide access to the site, the contractor had to rebuild 3½ miles of mountain trail road and build 1 mile of new road. As was the case at all shafts, to prevent pollution of the watershed the first major operation was the installation of a sewage treatment plant consisting of primary and secondary septic tanks, a dosing chamber, sand filter beds and facilities for chlorinating the effluent. The job water supply was pumped from a creek by a 25-gpm electric pump to a storage tank, then treated by a hypo-chlorinator. A power line was built to the site delivering current at 2,400 volts, stepped down to 220 volts and 110 volts by transformers at the job. Power consumption on this one job averaged 32,000 kwh. per month. Because of the remote location, the contractor built a camp, but it was never used, as the workmen preferred to live elsewhere and drive to work.

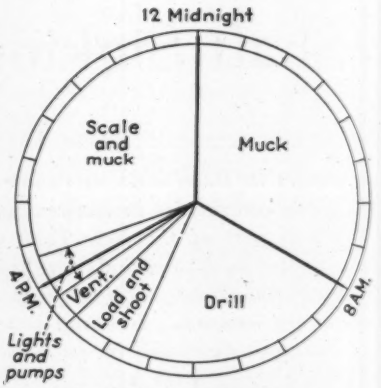
A powder magazine of double solid timber wall, separated by a 6-in. sand fill, and covered with corrugated metal, was built several hundred feet away from the shaft. On this particular job specifications permitted storage of 20,000 lb. of powder and 15,000 electric exploders, a greater amount than is allowed in populated locations.

A steel stiffleg derrick, steam-oper-



Powder: Cuts....5.4 lb. per hole
Others..4.3 " " "
Delays: Cuts.....0
Relievers..1+2
Trim.....2+3

Typical Drill Round
Shaft 2A



Typical Time Cycle
Shaft 2A

Fig. 3. Typical drill pattern and time cycle at shaft 2A. Dravo Corp. used two muck and one drill shift on this job; other contractors used mixed shifts, drilling or mucking as required.

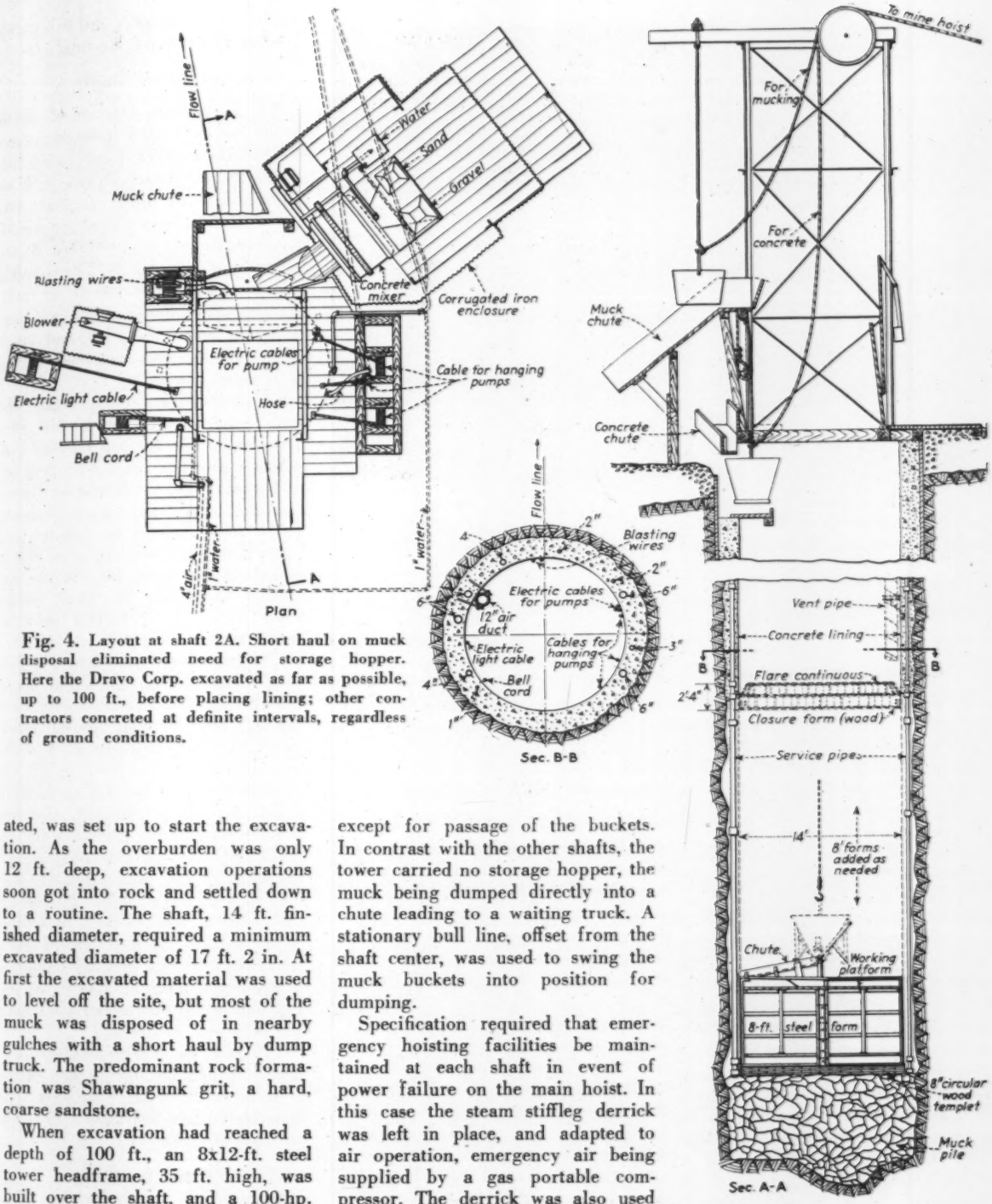


Fig. 4. Layout at shaft 2A. Short haul on muck disposal eliminated need for storage hopper. Here the Dravo Corp. excavated as far as possible, up to 100 ft., before placing lining; other contractors concreted at definite intervals, regardless of ground conditions.

ated, was set up to start the excavation. As the overburden was only 12 ft. deep, excavation operations soon got into rock and settled down to a routine. The shaft, 14 ft. finished diameter, required a minimum excavated diameter of 17 ft. 2 in. At first the excavated material was used to level off the site, but most of the muck was disposed of in nearby gulches with a short haul by dump truck. The predominant rock formation was Shawangunk grit, a hard, coarse sandstone.

When excavation had reached a depth of 100 ft., an 8x12-ft. steel tower headframe, 35 ft. high, was built over the shaft, and a 100-hp. electric mine hoist was installed. This tower happened to be one the Dravo Corp. had on hand, and was much less elaborate than the head frames designed and built for some of the other shafts. It carried a 4-ft. top sheave, and was fitted with a counterweighted platform, which, as a safety measure was used to close off the top of the shaft at all times except for passage of the buckets. In contrast with the other shafts, the tower carried no storage hopper, the muck being dumped directly into a chute leading to a waiting truck. A stationary bull line, offset from the shaft center, was used to swing the muck buckets into position for dumping.

Specification required that emergency hoisting facilities be maintained at each shaft in event of power failure on the main hoist. In this case the steam stiffleg derrick was left in place, and adapted to air operation, emergency air being supplied by a gas portable compressor. The derrick was also used for charging the hoppers of the concrete plant.

A concrete plant, consisting of a 1-yd. mixer, overhead sand and gravel bins and batching plant, was set up alongside the shaft. Concrete was discharged directly into a chute which loaded buckets set on a platform just below the surface level. A bull line spotted the buckets on the platform for loading operations.

Other top plant included the grout pump, a 3,000-cfm turbo-blower vent fan, two electric-drive stationary compressors, drill sharpening plant and furnaces and blacksmith shop. The turbo-blower ventilated the shaft through a 14-in. spiral-welded steel pipe carried to 60 ft. from the bot-

Fig. 5. Centrifugal booster pumps in chamber excavated at side of shaft 2. These were connected in tandem with pumps at bottom of shaft. Intake and discharge is through pipes set in concrete lining placed ahead of concreting.

tom. Usually ventilation was required only for a short time after shooting

Drilling and mucking

Drilling, loading and shooting procedure naturally varies under varying rock conditions and preferences of the superintendent. At shaft 2A a fairly uniform procedure was worked out as most suitable for the Shawangunk grit. A full round pattern of drilling was adopted instead of the benched bottom method used on some other shafts. Five wet jackhammers were used. Forged bits of 1-in. hexagonal steel were made up in lengths of 2, 4, 6, 7 and 9 ft. Bits were sharpened after each use.

The drill pattern consisted of eight or nine inclined cut holes about 9 ft. deep; two concentric rings of slightly inclined relief holes, 7 to 8 ft. deep, totaling from two to sixteen in number, and fourteen to twenty-two rib holes, inclined outward, 8 ft. deep. The average round consisted of 42 holes aggregating 268 ft. in depth. The lead wires of the exploders were connected to parallel bus wires, and the entire round fired together from an independent 220-volt line. Cut holes went off instantly, the inner ring of relievers on first delay, the outer relievers on second delay and the rib holes on the third delay.

Powder charges varied from ten to twelve sticks of 40 and 60 per cent gelatin per hole, averaging 187 lb. per round. In order to break up the top rock, a double prime system of loading was used, that is, from eight to ten sticks were placed in the bottom, with the primer stick second or third from the bottom, then 4 ft. of sand tamping was inserted, followed by two sticks of powder, the lower one primed. Very little top stemming was used. The rounds advanced the shaft an average of 5.1 ft. each.

Some contractors used the same crews for drilling and mucking, depending upon the work at hand. Dravo, however, believes it better to use separate drill and muck crews, splitting the day into three work shifts. From 8 a.m. to 4 p.m. the drill crew drilled out the round in 3 to 3½ hr., then loaded and shot. By the time the smoke cleared out it was usually 4 p.m. and the first muck crew came on; it installed the lights and pumps, scaled down and got a good start on the mucking. The second muck crew, working from midnight to 8 a.m., finished the job, leaving the hole clean for the drillers. All mucking was by hand, loading into side dump buckets. The drill crew consisted of five drillers, one helper, one powderman and a signal man. Muck crews included seven laborers below, one signalman, one top man and a truck driver. Two superintendents, three foremen, three hoist operators, one timekeeper, one electrician, one fireman, one blacksmith and two helpers and two pump operators completed the total crew.

Concrete lining

Not more than 100 ft. of depth could be excavated without placement of the concrete lining, a specification requirement. Dravo had no set schedule of lining at shaft 2A, but waited as long as ground conditions permitted, up to 100-ft. depth, before placing concrete. The concrete pours covered 40 to 100 ft. of shaft. No temporary lining or ground support was used. The drill and muck crews placed the concrete during their respective shifts.

In preparation for concreting, the last round of excavation was shot and leveled off, but not mucked out. A plank template carried the first lift of forms. Prior to placing of forms, the shaft walls were washed down, grout pipes placed in fissures and other required spots, and pans placed to deflect any water flow to the outside of the forms. The shaft service pipes were also extended prior to form setting.

The contractor designed his own steel forms of four collapsible quarter-sections in 8-ft. lifts. No reinforcing was required, but internal vibrators were used throughout. A working platform was carried at the top of each form lift, moved up as additional lifts were installed. Concrete was lowered to the platform in bottom-dump buckets, deposited around the edge of the platform and shoveled into the form. After an 8-ft. lift the concrete was allowed to set for 3 hr. before the next lift of form was added. Forms were available for a maximum pour of 100 ft. of lining, and no forms were stripped until the pour was complete.

Progress in placing lining averaged four lifts per day of three shifts, or about 50 ft. of shaft per week. The concrete volume averaged 3.25 cu.yd per foot of shaft.

Water handling

The engineers were determined that progress should not be delayed by groundwater troubles, specifying that whenever possible the water flow be grouted off in advance of excavation. When water was suspected or anticipated below, long holes were kept ahead of the normal drilling. As soon as water was discovered, excavation operations were stopped until the wet seam could be grouted.

TABLE I—CHARACTERISTICS OF DELAWARE AQUEDUCT SHAFT CONTRACTS

Contract	Shaft	Depth Ft.	Fin. Dia. Ft.	Predominating Rock
Cont. 332: $1,738,250 Dravo Corp.; A. C. Dennis, gen. supt.	2	825	14	Ashokan series sandstones, shales
	2A	1555	14	Shawangunk Grit
	3	840	14	Hudson River series shales, slates, sandstones
		3220		
Cont. 333: $2,294,415 Frazier-Davis Const. Co.; Don Blanks, mgr.; Fred Youmans, gen. supt.	4	877	19	" " " " " "
	5	1010	14	" " " " " "
	5A	1015	14	" " " " " "
	6	681	26½	" " " " " "
		3583		
Cont. 334: $1,926,150 W. E. Callahan Const. Co. & J. P. Shirley; H. L. Crawford, mgr.; Robt. Parker, J. R. Glaeser, gen. supts.	7	794	14	Highland Gneisses
	8	949	14	" "
	9 up	314	14	" "
	9 down	317	15	" "
	10 up	328	15	" "
	10 down	437	15	" "
		3139		
Cont. 336: $4,108,583 Dravo Corp.; R. W. Remp, gen. supt.; H. C. Richardson, asst.	17 up	320	15	Fordham Gneiss
	17 down	970	18	" "
	18 up	874	18	Manhattan Schist
	18 down	464	19½	" "
	19 up	449	19½	" "
	19 down	526	19½	" "
	20	432	16	Man. Schist and Inwood Limestone
	21	495	16	Manhattan Schist
	22	367	16	Fordham Gneiss
	23	322	22x32	" "
		5219		
Total all contracts: $10,067,398		15,161		

There was no standard procedure as to number and arrangement of grout holes or as to amount of grout and the pressure used. Each case was a matter of judgment and experience on previous cases. For this work Dravo used its Francois Cementation System of grouting. The grout plant—a double acting air pump and two 100-gal. mixing tanks—was kept on the surface, thus adding the hydrostatic head in the pipe to the pump pressure. Pressures varied from 120 to 700 lb. per sq.in. The grout feed pipe was left in place outside the concrete lining. The worst water trouble at shaft 2A occurred 200 ft. below the surface when a flow of 130 gpm developed in a mud seam after excavation had passed 20 ft. below that point.

Since it was impossible to stop all leakage into the shaft by grouting, plenty of pump capacity was provided. A 50-gpm low-head electric drive self-priming centrifugal pump was kept on the bottom of the hole. This pump discharged directly into a high-head centrifugal pump, also electric drive, suspended from cables close to the bottom, but high enough to be out of the way. Both pumps were removed during blasting.

When a depth of 475 ft. was reached, a pump chamber and sump was excavated back from the shaft, and an electric centrifugal pump installed. This was intended to serve as a booster for the two pumps at the bottom, but on this shaft so little water was found below the 500-ft. depth that the bottom pumps were no longer necessary. Instead, the little water that did collect on the bottom was bailed out. All leakage from above the pump chamber was deflected by a water ring to the sump at the chamber level. Incidentally, after excavation had passed the pump chamber, water for the drills was obtained from the pump sump, the pressure being regulated by a valve.

Installation of service pipes

Prior to placement of each section of concrete lining, the ten service pipes had to be extended to the bottom. As some of these pipes were 6-in., up to 100 ft. long, a special handling procedure was required. At shaft 2A a circular platform, narrower than the finished shaft, was hung from tiebolts embedded in the lower end of the last previously placed concrete. The first 20-ft. length of pipe was capped with a regular coupling and a special sleeve containing a lifting eye, and lowered through a hole in the platform until a collar caught the top coupling. The sleeve was then removed and another pipe section lowered to the platform, coupled and the assembly dropped again to the top coupling. This procedure was repeated until the full length of pipe desired was assembled. Meanwhile, a 24-in. nipple was attached to the pipe already embedded in the wall as an extension to clear the bottom of the concrete. Then the assembly of new pipe was hoisted clear of the platform and lowered alongside through a timber yoke until the top coupling rested against a collar on the yoke. While the pipe was supported from the bottom, the hoist line was transferred to a lifting eye close to the collar in the yoke, and the sleeve removed from the pipe. Then the yoke and its load was carefully hoisted, steered by men on the platform to bring the new and old pipes together for coupling.

After the shaft had been sunk to desired depth, headings were turned for 200 ft. each way on the tunnel line as part of the shaft contract. Because of the short distance, the contractor did not organize or equip for tunnel driving, but worked the heading-and-bench method by drifter drilling from columns and bars; mucking was by hand. The bottom of the shaft and tunnel sections were supported by steel timbering.

Direction

The Delaware Aqueduct project is being built by the New York Board of Water Supply, of which Walter E. Spear is chief engineer, Charles M. Clark, deputy chief engineer, and Roger W. Armstrong, department engineer in charge of design. The project has been divided into three geographical departments: Eastern, Harry R. Bouton, department engineer; Northern, Neil C. Holdredge, department engineer; Reservoir, James A. Guttridge, department engineer. The contractors and their superintendents on the shaft work are given in Table I.

Bay City Improves Its Bridges

CRAIG P. HAZELET

Hazelet & Erdal, Consulting Engineers, Chicago, Ill.

Contents in Brief—After half a century of service the bridges of Bay City, Mich., required drastic rehabilitation or replacement. Improvement work recently accomplished includes one new bridge and reconstruction of two existing ones to carry modern loads.

IN BAY CITY, MICH., crossing of the Saginaw River, which cuts through the city close to the business section, was until recently effected by four bridges more than 40 yr. old, of which three were grossly inadequate for modern traffic. The fourth bridge, reconditioned in 1938, was capable of carrying H-15 loads and was therefore satisfactory. Replacement of one of these old structures by a new bridge and extensive though inexpensive repair of two others have greatly improved the city traffic situation.

In 1934 the three inadequate bridges were posted for maximum loads varying from 1½ to 8 tons. This was followed by condemnation of one of the structures, the Lafayette Ave. bridge, an action which focused the attention of the city on the pressing question of bridge replacements. However, no progress was made because of financial difficulties, for two years, when a city-wide bridge survey was authorized. The program evolved from these studies was as follows: Immediate construction of a temporary pontoon swing span for the Lafayette Ave. structure, followed by construction of a new Lafayette Ave. bridge; rehabilitation of the Belinda St. bridge; and repairs to the Cass Ave. bridge including replacement of the east channel swing span, this bridge and the one at Lafayette Ave. crossing over an island in the river which creates an east and west channel.

Lafayette Ave. pontoon

For 3½ years after the Lafayette Ave. bridge was condemned until the new one was put in service late last year a pontoon bridge did duty over the navigated channel; it was flanked on both ends by timber trestle approaches. This structure had a 20-ft. roadway and one 5-ft. sidewalk and was designed for H-15 loading. The movable truss span, 135 ft. long, was supported on one end by a timber pier, where one truss rested on a spherical bearing and the other on a two-wheel truck running on a circular track. The front end of the span was supported on a 26x56x8-ft. barge at the second and third panel points. When the span was in the closed position the barge was lowered by pumping in water ballast until the span rested on the front pier. Thus the structure rested firmly on the supporting piers when carrying traffic. When it was desired to open the span, water ballast could be removed in 60 sec. to lift the span clear of the rest pier. A gasoline-driven single-speed hoist, mounted on a barge with front and back haul cables anchored to a pile cluster, operated the pontoon. The barge was divided laterally into three watertight compartments to obtain lateral bracing and to reduce the surging effect of the water ballast, ordinarily placed in the center compartment only. The cost was $37,500.

New Lafayette Ave. bridge

At Lafayette Ave. the river is 1,500 ft. wide with a 500-ft. island in the center. Over the east branch, 18½ ft. deep and navigable for a 200-ft. width, a 185-ft. double-leaf skew bascule, affording 150-ft. horizontal navigation clearance was erected in 11 months and put in service last November. Over the non-navigable west channel a new fixed bridge of deck girder spans with curved bottom chords was constructed. The bascule is flanked on the east by a deck plate

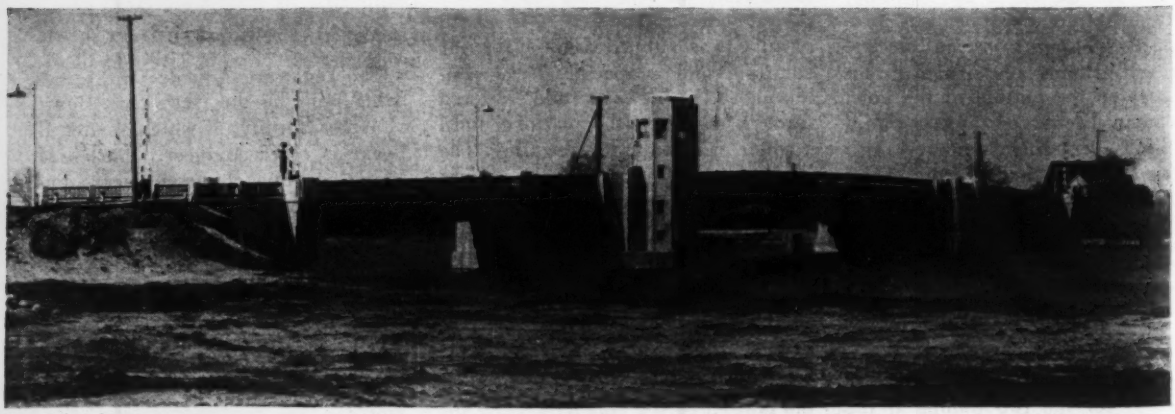


Fig. 1. New Lafayette Ave. bridge at Bay City, Mich. This bascule, across the east channel of the Saginaw River, was recently designated the most beautiful movable bridge of the year.

girder span, 107 ft. 8 in. long, and on the west by a plate girder span 96 ft. 10 in. long. The approach spans accommodate a 30-ft. roadway on a 7½-in. concrete slab and two 5-ft. sidewalks on a 4-in. slab. To reduce weight, the 30-ft. roadway of the bascule span is made of 3½-in. grid-type decking, filled with light-weight concrete, while similar construction with 2½-in. grid section is used for the sidewalks. Weight per square foot of deck is 45 lb. for the roadway and 25 lb. for sidewalks.

The bascule span, which received the A.I.S.C. award for the most beautiful movable bridge of the year (*ENR*, June 22, 1939, p. 39), is of the Scherzer rolling lift type with the rear break in the floor in front of the point of support with bridge closed. Each leaf of the bridge is operated by two 25-hp motors. Emergency brakes of the thrustor type are provided, and the total braking capacity is more than ample to hold the bridge in the open position against a wind load of 15 lb. per sq. ft. The motors are located on the movable span and drive through a special speed reducer unit, equipped with built-in equalizer, thence through two open gear trains to the fixed racks. The use of the speed reducer unit simplifies the machinery layout and facilitates access to all parts of the equipment. Both leaves of the bridge, as well as the traffic gates and signals, are operated from one operator's house located at the east bascule pier.

On a floor at the track girder level is the auxiliary or standby power equipment consisting of a 75-kva generator, direct-connected to a 120-hp 1,200-rpm gasoline engine. To change from the main power supply to auxiliary power requires only the operation of three "throw-over" switches.

The total cost of the project, including both east channel and west channel bridges, was $618,000.

Belinda St. bridge

The 47-yr. old Belinda St. bridge, serving the north end of the city, consists of one 260-ft. through truss swing span flanked by a 140-ft. through truss span at each end. Two approach girder spans made the total length of bridge 600 ft. All metal above the deck was in satisfactory condition but below deck corrosion was well advanced, particularly on the floorbeams, stringers and sidewalk brackets. The bottom chords of the center panel, immediately over the pivot pier, were likewise corroded to a dangerous degree. A generous use of salt at the approaches during slippery weather had obviously done much to accelerate the deterioration of the metal. Finally, dredging of the channel had robbed the pivot pier of its original stability, and frost action had taken a heavy toll at the joints in the rubble masonry of all piers.

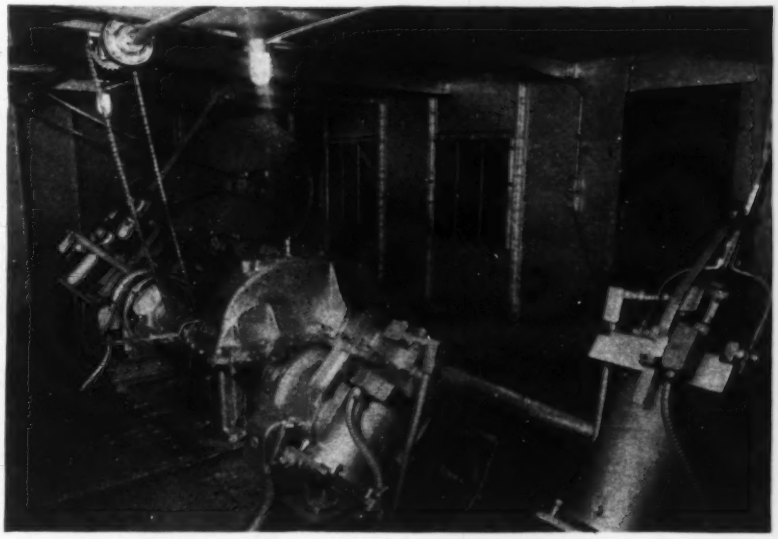


Fig. 2. Operating machinery for the Lafayette Ave. bascule. Motors drive through a speed reducer unit in the housing near the center of the picture.

Calculations indicated this structure, the replacement value of which was $500,000, could be reconditioned to carry H-15 loading for $80,000. WPA did the job. The items included were as follows: (1) stabilization of pivot piers and rest piers by steel sheetpiling; (2) replacement of fender system; (3) deck replacement with concrete-filled steel grid; (4) new treated-timber sidewalks; (5) new electrical controls, electrical brakes, roadway gates, signals and operator's house.

Cass Ave. bridge

Salvage from the Lafayette Ave. temporary bridge, including the pontoon span, is being used this season to recondition the Cass Ave. bridge and to widen the channel from 67 to 100 ft. This work involves the following items: (1) removal of the east swing span; (2) dredging; (3) moving in the pontoon span from Lafayette Ave.; (4) replacement of the timber approach to the west channel swing span, using salvaged material; (5) strengthening the floor system of the west swing span and replacement of the deck with salvaged material.

W. J. Meagher, Bay City, Mich., had the contract for the Cass Ave. bridge at $23,000. The Owens-Ames Kimball Co., Grand Rapids, Mich., with Max Pearse as general superintendent, was general contractor for the Lafayette Ave. bridge; R. C. Mahon Co., Detroit, was sub-contractor for steel fabrication and erection. A. L. R. Sanders was resident engineer for Hazelet & Erdal, consultants for the entire program. J. Harry Nelson, city manager, represented the city in all negotiations.

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Thousands Complete Courses for WPA Foremen

More than 46,000 foremen on WPA construction projects have completed on their own time special training courses offered by the WPA. Whereas there were originally courses in the supervision of labor, the program has now been expanded to include classes in semi-technical phases of construction and road-building, and about 19,000 men are now enrolled in such study.

The courses in supervision include such topics as discipline, giving orders, planning project work, and accident prevention. The semi-technical courses include interpretation of drawings, measurements of quantities, soils, road surfacing, concrete work, scaffolding and rigging.

FROM FIELD AND OFFICE

Working problems and time-saving methods . . . Edited by Charles S. Hill

A Self-Flushing Culvert

To prevent the lodgement of debris in railroad culverts, the Southern Pacific Co. makes use of a culvert design in which the floor is built on a slope steep enough to insure ample velocity for sluicing out debris brought into the structure by floods.

A typical "flushout" culvert for a single line of track. Picture taken before the roadbed was graded.

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Solving Electrolysis Trouble

F. M. Carver
Maintenance Foreman,
The Reliance Electric & Engineering Co.,
Cleveland, Ohio

Due to stray electrolytic currents and soil of a corrosive chemical nature, several sections of an 8-in. plain cast-iron water main of the Reliance Electric & Engineering Co. of Cleveland, Ohio, developed severely corroded pits in their surfaces and eventually leaked to the extent that the sections had to be replaced. Although only a few sections were affected it was evident, because of similar unfavorable soil conditions all along the main, that leakage might be expected at other points unless all the pipe sections were treated.

To remedy these conditions it was decided to remove all the soil within at least 1½ ft. of the pipe, which lay from 4 to 8 ft. underground. The pipe sections were then connected by a bare No. 6 copper wire running the whole length of the main. This wire was attached to each section with ground clamps and carefully grounded at both ends of the line. The pipe surface was painted with hot asphalt, then wrapped with asphalt-impregnated paper and painted again with hot asphalt. The soil which had immediately surrounded the water main was completely replaced with a dry sand loam, secured from a different locality.

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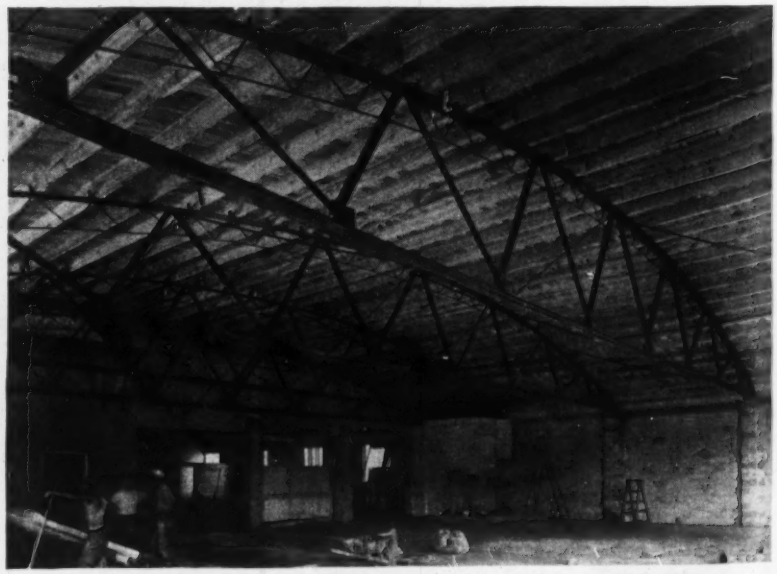
Unique Bowstring Truss

John Graveley
Architect, Jacksonville, Fla.

An unusual type of welded steel bowstring truss has been used in a telephone building at Orlando, Fla., which it is believed effects a substantial saving over more conventional designs. This saving is effected largely by the use of a top chord consisting of a 6x⅜-in. plate laid flat and easily bent to radius, which is stiffened between panel points by an open-web bar-joist. This design, incidentally, is applicable to other than bowstring trusses.

The lower chord is a 5 in. by 6.7 lb. channel while diagonal web members consist of single angles 2½x2½x¼ in. The angles connect to the lower chords through gussets but are welded directly to the top chords. These web members are stressed only under an unbalanced live load. The trusses have a 60 ft. span and a 6½ ft. depth at the center. They were designed by the writer and erected by R. L. Graveley, Inc., steel contractors. Earl Chabot was the contractor on the building.



Light, economical bowstring roof trusses feature a flat plate top chord braced by an open web bar joist detail.

CAMERA MEMORANDA

Freshly poured concrete in Grand Coulee Dam is dry enough so that ordinary boots suffice for the workmen who operate the vibrators. However, in some locations, and under special conditions, it was found desirable to provide wooden sandals to enable workmen or inspectors to walk over the fresh surface without sinking in. The sandals are made of boards with rounded ends, provided with wire loops into which the toes of shoes can be tucked readily.

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CONTRACT UNIT PRICES

What contractors are bidding on different kinds of construction work

HIGHWAYS, RHODE ISLAND

OWNER: State of Rhode Island, Department of Public Works, Division of Roads and Bridges.

PROJECT: Construction of Nooseneck Hill Road, Towns of West Greenwich and Exeter, in Kent and Washington Counties, R. I. Dual type, reinforced cement concrete and bituminous macadam penetration. Width to be 47 ft.; reinforced cement concrete in outside lanes 11 ft. each; bituminous macadam inner lane, 25 ft. wide. Pavement thickness: 8 in. Length, 3.801 miles.

CONDITIONS: Labor rates are: skilled, 75c. per hr.; semi-skilled, 60c. per hr.; and common, 50c. per hr. Work is to be started September 5, and to be completed by August 1, 1940. Contractor to furnish all materials. Highway transportation available.

BIDS: Six bids were received on June 5, 1939, ranging from the contract low of $288,169 to $399,387.

LOW BIDDERS:

1. M. A. Gammino Constr. Co., Providence, R. I. $288,169
2. Callan Constr. Co., Inc., Bristol, R. I..... 295,946
3. Arute Bros., Inc., New Britain, Conn..... 306,661

Item	Quan.	Unit Prices (1)	(2)	(3)
1. Clear and grub	9.66 ac.	$75.00	$60.00	$75.00
2. Cut and remove trees	53 ea.	8.00	8.00	20.00
3. Earth excav	79,313 c. y.	.18	.20	.30
4. Trench excav., under 5 ft.	5,844 c. y.	.60	.50	.75
5. Trench excav. 5–10 ft	35 c. y.	1.00	.60	1.13
6. Rock or ledge excav	13,158 c. y.	.80	1.25	.30
7. Trench ledge excav. und. 5 ft	830 c. y.	2.00	2.50	.45
8. Trim and fine grading	204,650 s. y.	.02	.02	.03
9. Strip and pile loam	2,449 c. y.	.20	.35	.30
10. Replace loam	2,449 c. y.	.40	.40	.50
11. Furn. and pl. loam	8,434 c. y.	.60	.60	.80
12. Furn., place explosives	3,000 lb.	.20	.27	.30
13. Gravel foundation, "A"	51,638 c. y.	.37	.35	.30
14. Cr. stone or gravel base crse	13,937 ton	1.30	1.45	1.40
15. Filler, base crse	3,484 ton	1.00	1.60	1.50
16. Gravel base crse	7,329 c. y.	.42	.35	.30
17. Cr. grav. top for bitum. mac	10,035 ton	2.30	2.48	2.75
18. FHH&A Bitum. binder	139,375 gal	.07	.08	.08
19. Rein. cem. conc. pavmt	10,902 c. y.	3.50	3.25	3.75
20. Cement, conc. pavmt	17,956 bbl.	2.35	2.34	2.40
21. Steel bar reinf. (untr.)	14,296 lb.	.04	.04	.05
22. Steel bar reinf. (tr.-support-sleeve)	18,044 lb.	.08	.05	.08
23. Steel fabric reinf	52,166 s. y.	.14	.16	.16
24. Premix type H, cl. C. surf.	43,976 s. y.	.23	.20	.25
25. FHH&A Bitum. mat'l., MC-3	62,385 gal.	.07	.08	.08
26. 8-in. sub-drain	1,000 l. f.	1.00	1.25	1.00
27. 12-in. sub-drain	2,610 l. f.	1.50	1.50	1.25
28. 12-in. Cl. A rein. conc. pipe	1,308 l. f.	1.10	1.10	1.20
29. 18-in. Cl. A rein. conc. pipe	55 l. f.	1.75	1.80	2.00
30. 24-in. Cl. A rein. conc. pipe	90 l. f.	2.75	2.80	3.00
31. 12-in. Cl. B corr. steel pipe	4,948 l. f.	.60	.65	.75
32. 18-in. Cl. B corr. steel pipe	2,072 l. f.	1.30	1.30	1.40
33. 24-in. Cl. B corr. steel pipe	1,044 l. f.	2.45	2.15	2.50
34. 30-in. Cl. B corr. steel pipe	22 l. f.	3.75	3.85	4.00
35. 8-in. Cl. B corr. steel pipe	94 l. f.	.50	.45	.50
36. 12-in. lock joint C. I. P	416 l. f.	2.00	1.40	2.20
37. 18-in. spir. corr. C. I. P	551 l. f.	2.50	1.90	3.50
38. 18-in. lock joint C. I. P	18 l. f.	2.50	2.10	4.00
39. 24-in. spir. corr. C. I. P	9 l. f.	5.00	5.00	6.00
40. Constr. drop inlets	8 ea.	50.00	60.00	60.00
41. Remov. and relay 12-in. C. I. P	114 l. f.	.60	.60	1.00
42. Remov., relay 18-in. spir. corr. C. I. P	144 l. f.	.60	.80	1.00
43. CB "D" comb. H&B, with precast conc. inlet	8 ea.	35.00	25.00	40.00
44. CB "D" comb. H&B gutter inlet and LeBaron	66 ea.	45.00	33.00	40.00
45. 8-in. vert. wall of comp. peri	435 v. l. f.	10.00	7.50	6.00
46. Remov. and backfill abandoned C. Bs	11 ea.	2.00	8.00	10.00
47. Cem. conc. masonry	75 c. y.	15.00	15.00	10.00
48. Steel bar reinf	1,058 lb.	.05	.06	.06
49. Stone masonry	361 c. y.	8.00	8.00	10.00
50. Remov., rebuild stone walls	8,349 l. f.	.55	.50	.50
51. Cem. conc. edging	11,109 l. f.	.40	.35	.50
52. Premix. bit. conc. walk, no found	3,802 s. y.	.60	.80	.80
53. Set fence posts	418 ea.	.75	1.85	1.60
54. Erect wire fencing	4,909 l. f.	.15	.15	.15
55. Guard rail, wood, type N	6,351 l. f.	.34	.40	.40
56. Struct. excav., earth, br	226 c. y.	2.00	3.00	2.00
57. Channel excav., earth, br	70 c. y.	1.00	2.00	1.00
58. Masonry excav., br	20 c. y.	4.00	4.00	7.00
59. Cofferdam, place, cut (br.)	3,240 s. f.	.60	.15	.30
60. Class A conc., bridge	281.5 c. y.	16.00	18.00	16.00
61. Stone masonry, bridge	43 c. y.	20.00	25.00	10.00
62. Conc. surf. finish (rubbed)	290 s. f.	.15	.10	.12
63. Conc. surf. finish (gran. walk)	54 s. y.	.70	2.00	1.00
64. Reinforcing bars	21,726 lb.	.04	.05	.06
65. Joint fill r. bridge	385 s. f.	.30	.15	.20
66. Bitum. waterproofing, br	216 s. y.	.60	.20	.50
67. Grav. fill (sp. spandrel), br.	80 c. y.	.50	1.00	1.00
68. Rock fill, bridge	63 c. y.	1.50	3.00	2.50
69. Remove, dispose exist. R. C. C. pavmt	7,779 c. .	.75	.60	.50
70. Spec. dry masonry wall	143 c. y.	12.00	10.00	10.00
71. Seeding (slope areas)	43,216 s. y.	.04	.05	.05
72. Plant vines	14,882 ea.	.12	.12	.15
73. Seed (slope mix A)	1,341 lb.	.30	.27	.35
74. Fertiliser (8–6–4)	9,725 lb.	.02	.021	.03

PENNSYLVANIA TURNPIKE

OWNER: Pennsylvania Turnpike Commission, 11 N. Fourth St., Harrisburg, Pa., S. W. Marshall, engineer.

PROJECT: Construction of 1.7 miles of Turnpike roadway, Section 19D, Contract 39, in Hopewell, Lower Mifflin, Upper Mifflin and Upper Frankford Townships, Cumberland County, Pa., consisting of grading, drainage, pavement and construction of reinforced concrete structures.

CONDITIONS: Time for completion, 260 days. Contractor to furnish all materials. Labor rates are: skilled, $1.00 to $1.50; semi-skilled, 65c. to $1.00; and common, 52½c. per hr.

BIDS: Eleven bids were received July 20, 1939, ranging from the contract low of $886,027 to $1,108,220.

LOW BIDDERS:

1. Walker Bros., Chambersburg, Pa. (contract) $886,027
2. Union Paving Co., Philadelphia, Pa..... 928,840
3. Collins & Maxwell, Inc., Easton, Pa..... 936,531

Item	Quan.	Unit Prices (1)	(2)	(3)
1. Clear and grub	L. S.	$3,000.00	$1,000.00	$1,000.00
2. Class 1 excav	39,640 c. y.	.35	.35	.30
3. Class 2 excav	2,556 c. y.	1.50	1.70	2.00
4. Borrow excav	16,588 c. y.	.30	.35	.30
5. Shape roadway	48,555 l. f.	.15	.10	.174
6. Subgrade	288,312 s. y.	.07	.10	.10
7. Shoulders	54,055 l. f.	.15	.10	.15
8. Cem. conc. pave., 12-in. deep	22 s. y.	2.40	3.00	3.00
9. Reinf. cem. conc. pave., 9-in. deep	288,290 s. y.	2.35	2.49	2.53
10. Reinf. conc. wear. surf., br. floors	1,635 s. y.	2.25	2.00	2.00
11. Class A concrete	569 c. y.	20.00	22.00	21.00
12. Class B concrete	1,658 c. y.	17.00	17.00	17.00
13. Plain steel bars	216,516 lb.	.04	.04	.04
14. Stone backfill, misc. drain	64 c. y.	2.75	2.50	3.00
15. Stone backfill, struct	44 c. y.	2.75	2.50	3.00
16. Cement rubble masonry	18 c. y.	12.00	14.00	16.00
17. Type RS inlets	38 ea.	80.00	100.00	100.00
18. Type R inlets	8 ea.	80.00	100.00	100.00
19. Corr. metal pipe, 12-in.	56 l. f.	1.75	1.50	1.25
20. Corr. metal pipe, 15-in.	330 l. f.	2.00	1.80	1.75
21. Corr. metal pipe, 18-in.	222 l. f.	2.50	2.00	2.25
22. Corr. metal pipe, 24-in.	82 l. f.	3.00	2.60	3.00
23. Corr. metal pipe, 36-in.	36 l. f.	5.50	4.50	5.50
24. Asph. coat. corr. met. pipe No. 8Ga. (F. C.), 54-in.	158 l. f.	12.50	13.00	15.00
25. Reinf. cem. conc. pipe, 18-in.	960 l. f.	3.00	3.00	3.00
26. Pl. cem. conc. or vit. cl. pipe, 18-in.	1,645 l. f.	2.25	2.25	2.25
27. Corr. met. pipe relaid, 12-in	16 l. f.	1.00	1.50	1.00
28. Perf. vit. cl. tile underdrain, 6-in	29,238 l. f.	.80	.75	.70
29. Perf. vit. cl. tile underdr. lat. 6-in.	10,560 l. f.	.90	.75	.70
30. Perf. vit. cl. tile fnd'n. underdr, 4-in.	1,150 l. f.	.80	.70	.60
31. Tile outlets, 4-in	352 l. f.	.70	.60	.60
32. Tile outlets, 6-in	1,512 l. f.	.75	.70	.70
33. Subgrade drains	2,750 l. f.	.40	.20	.30
34. Type P. T. F. guard fence	41,865 l. f.	.75	.75	.748
35. Type 1-C guard fence	720 l. f.	.50	.40	.40
36. Type P. T. C. intermediate take-up assemblies	23 ea.	37.50	35.00	33.00
37. Type P. T. F. end anchorages	130 ea.	22.00	22.00	16.00
38. Type C end anchorages	8 ea.	12.50	10.00	10.00
39. Top soil borrow	9,211 c. y.	.45	.60	.25
40. Approved shale borrow	13,676 c. y.	.35	.40	.25